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LEADING CONTENTS

	PAGE
The "Trades" of Architecture and Engineering	1
Specification for Concrete Work. *By G. P. Manning, M.Eng.*	3
Reinforced Concrete Residential Flats at Edinburgh	9
Concrete Construction in the Tyne Tunnels	15
A Continuous Prestressed Concrete Girder Bridge in France	21
Piles 87 ft. long at Plymouth	28
Reinforced Concrete Grandstands	31
Reconstruction of Quay Wall, Rotterdam	33
An Unusual Reinforced Concrete Building	35

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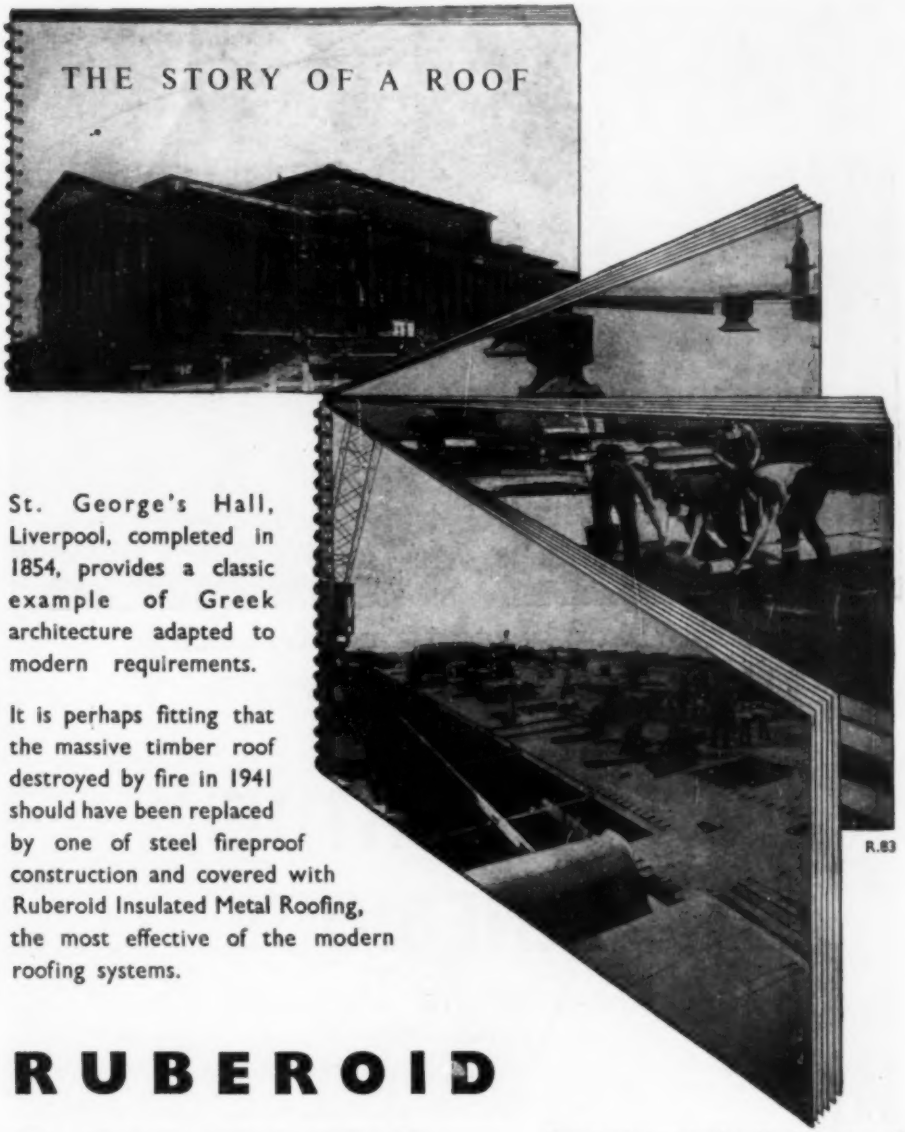

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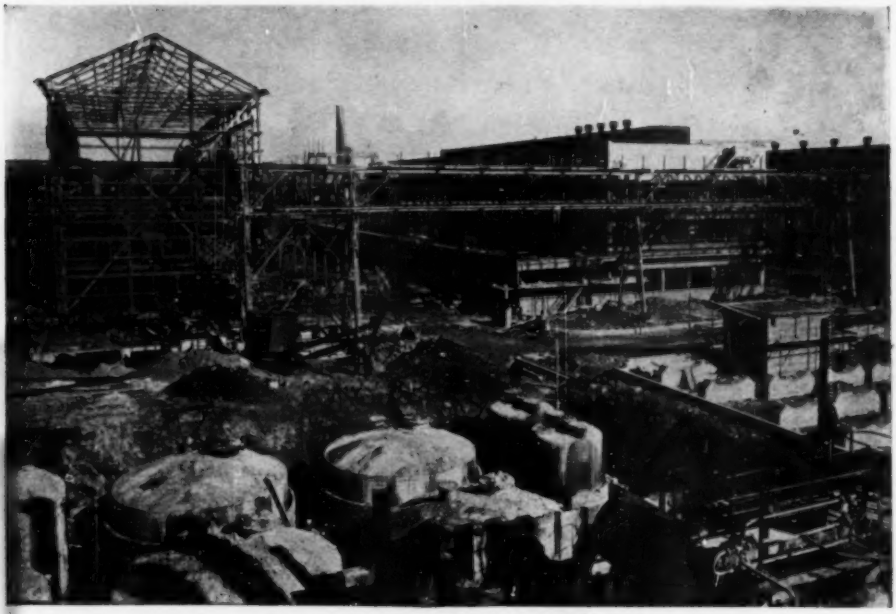

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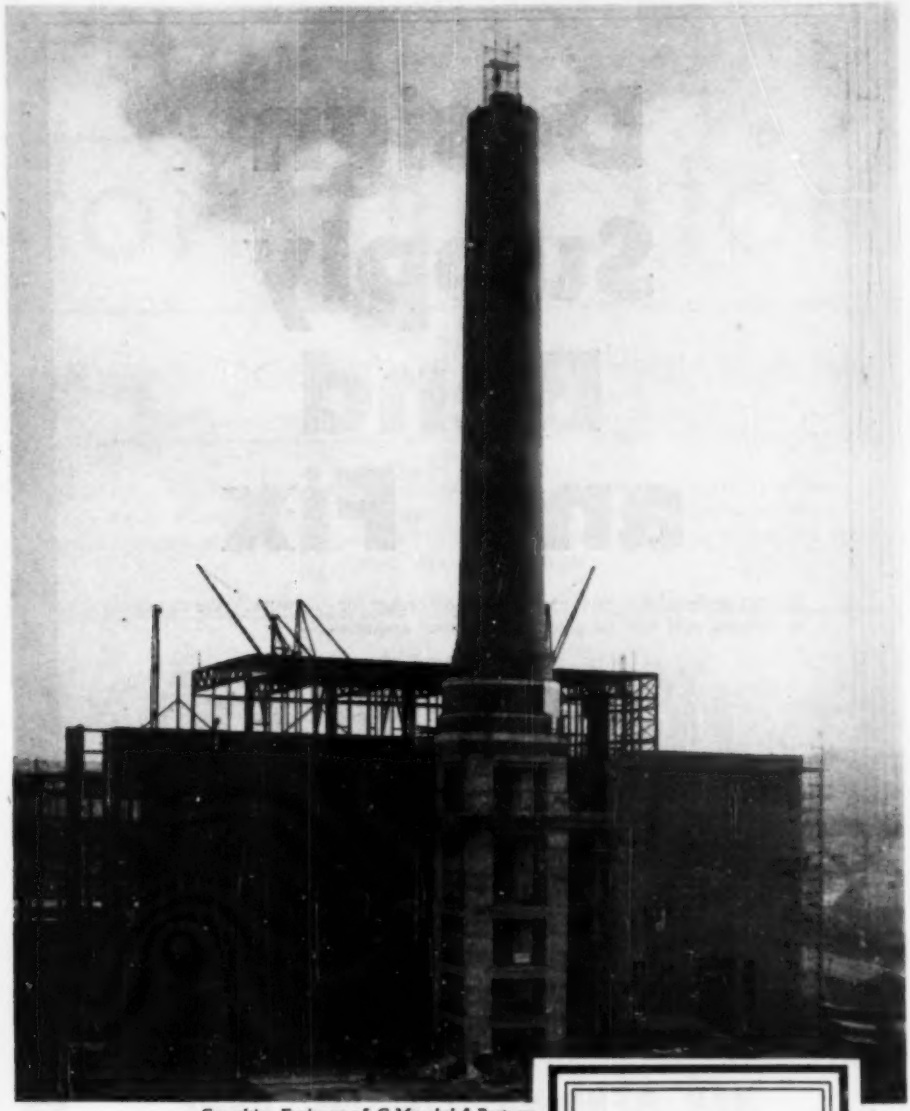

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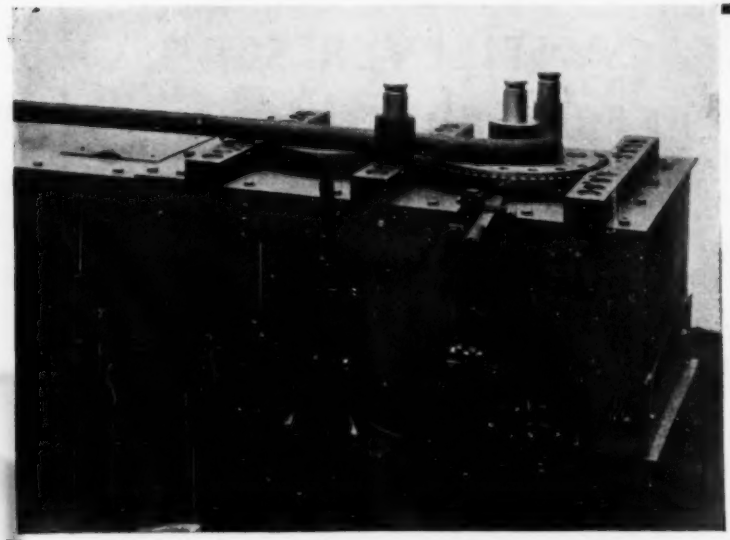

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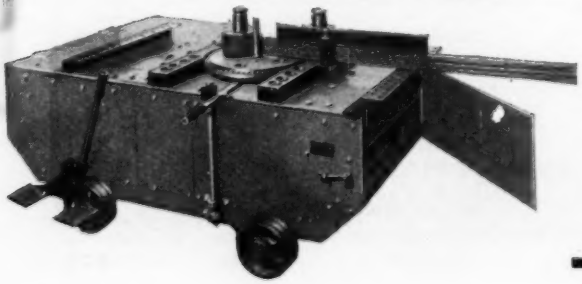

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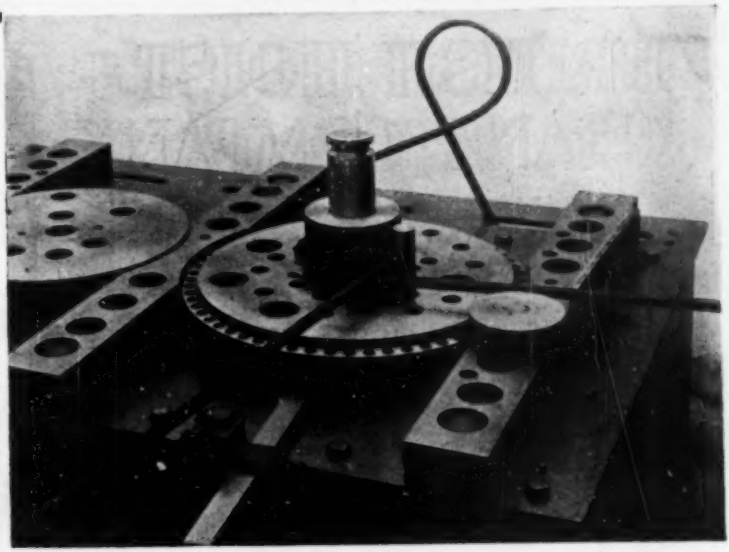

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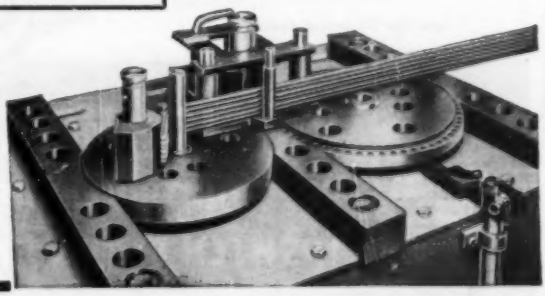

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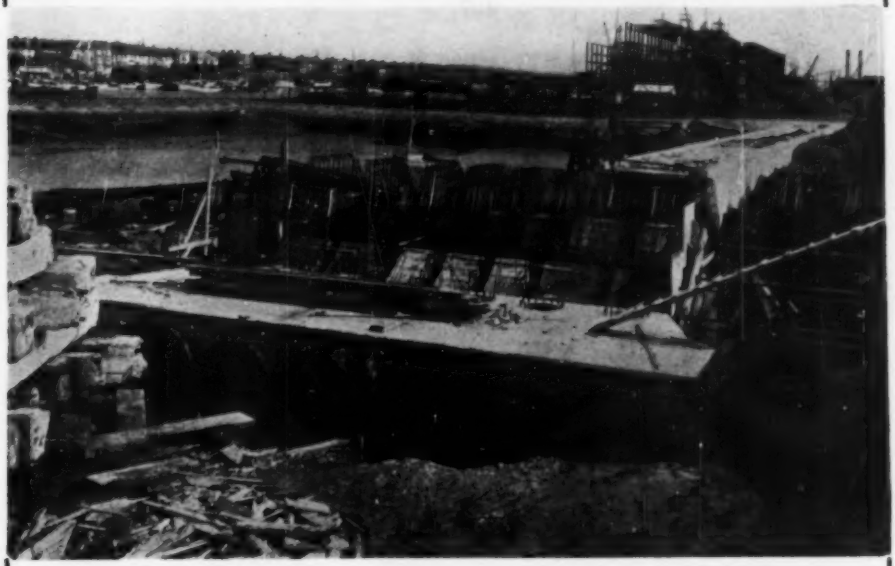

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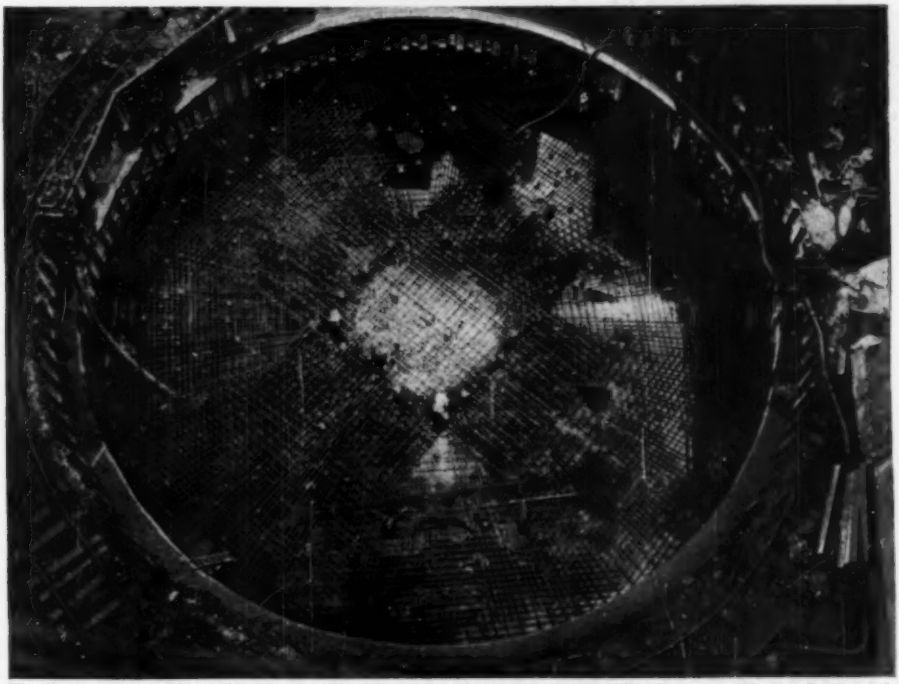

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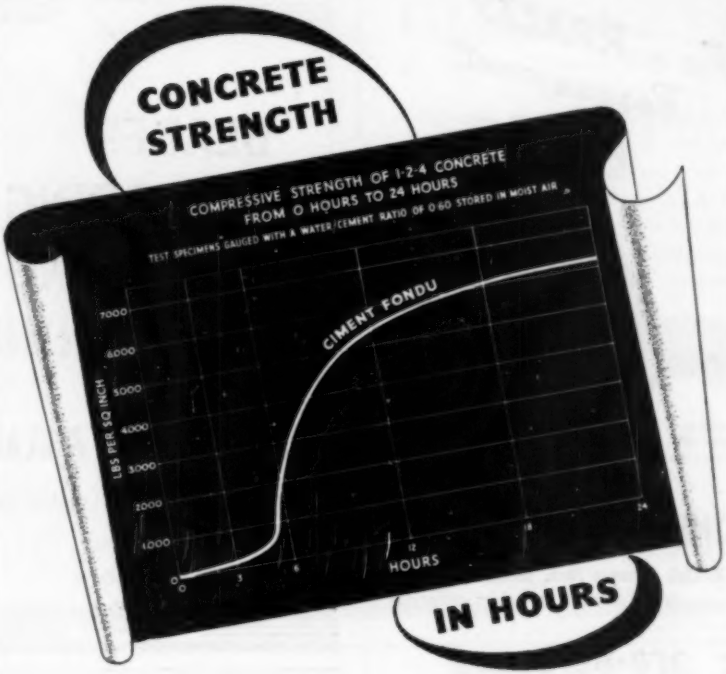

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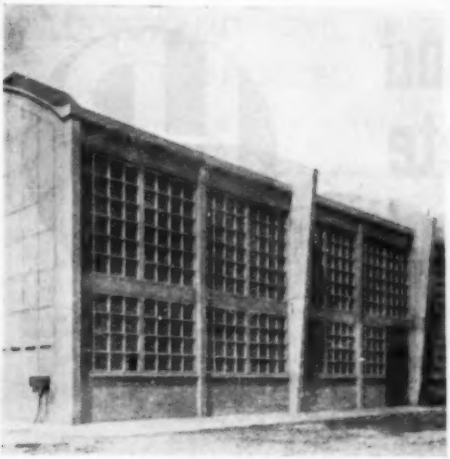

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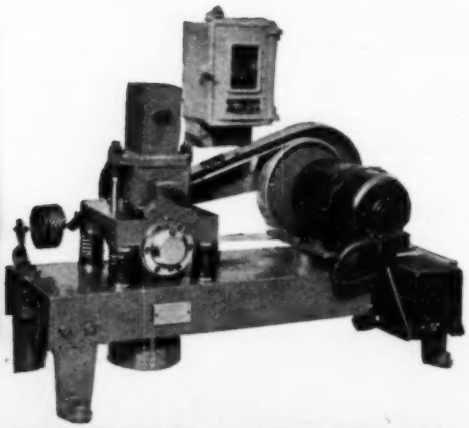

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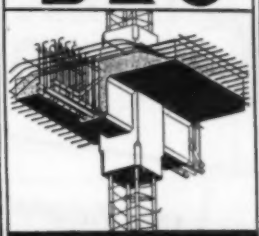

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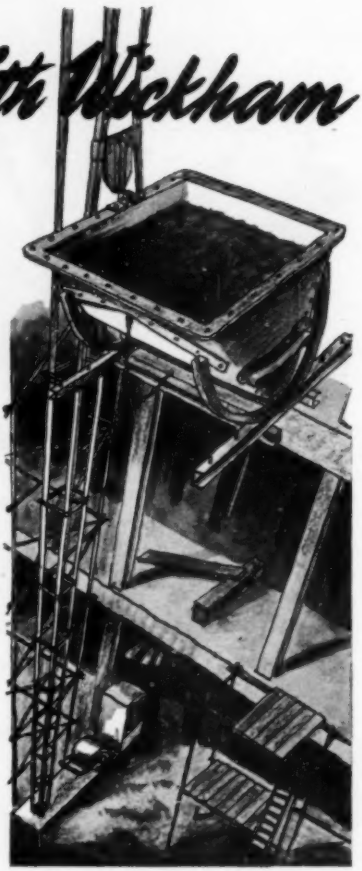

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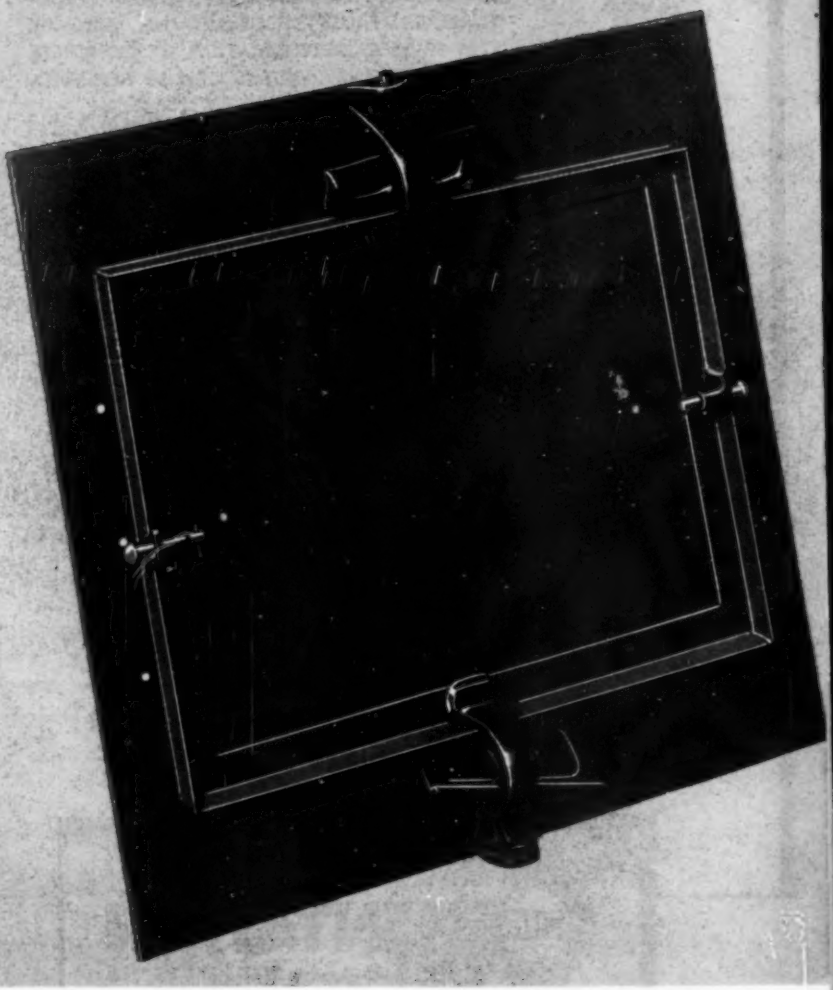

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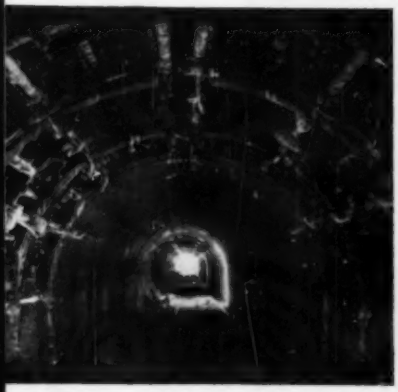

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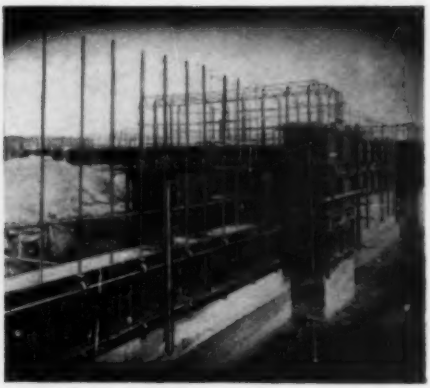

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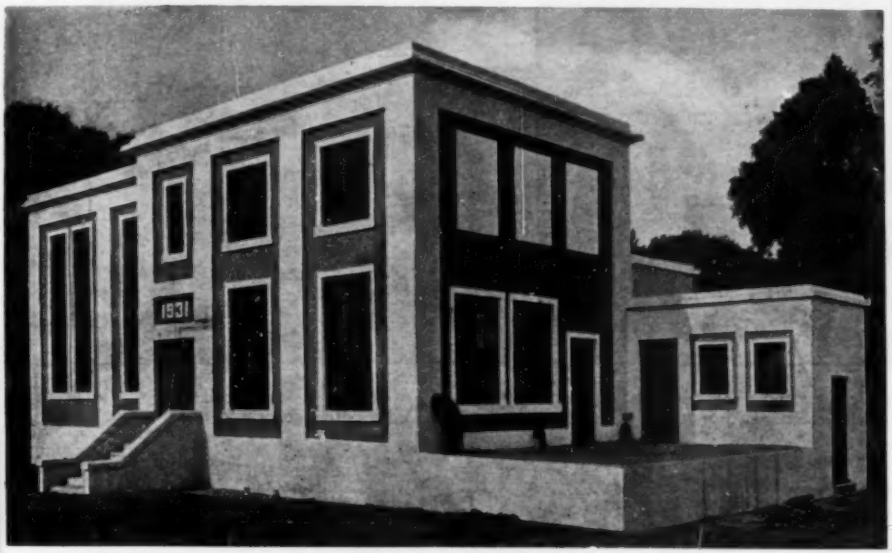

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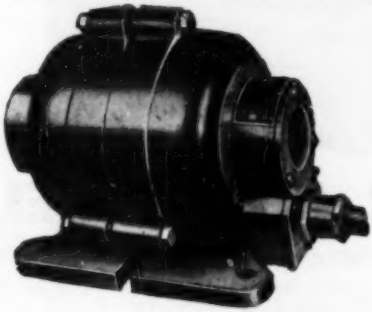

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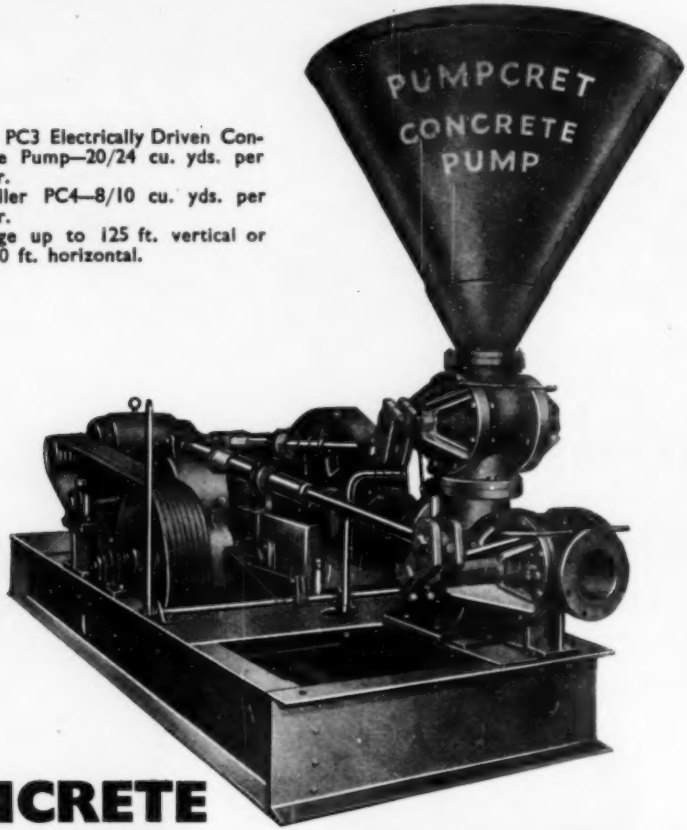

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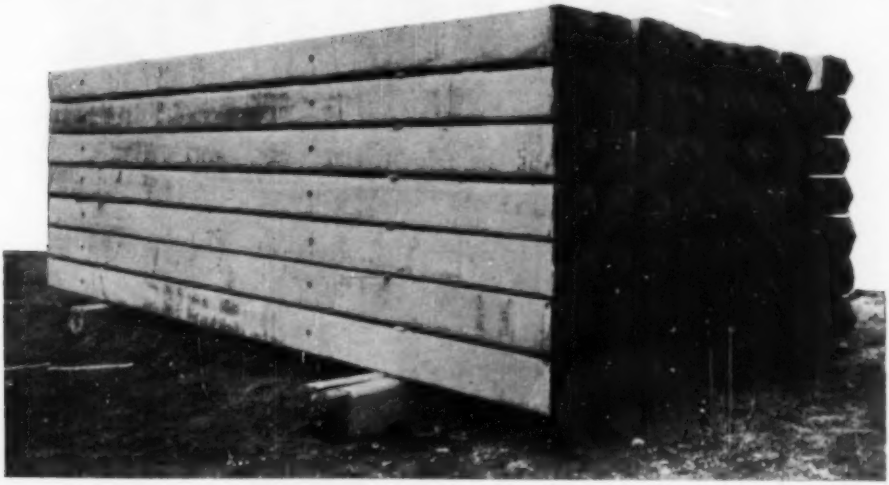

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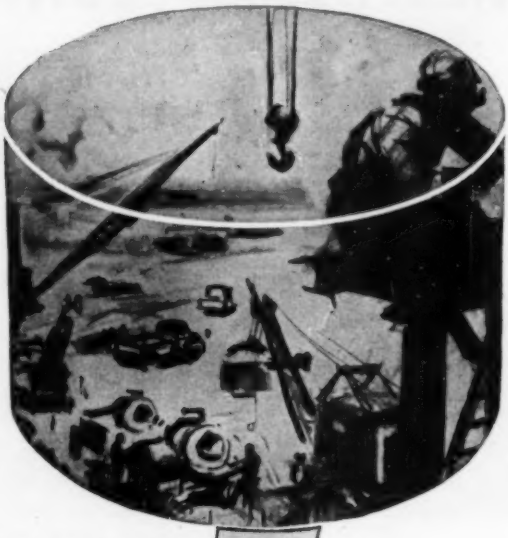

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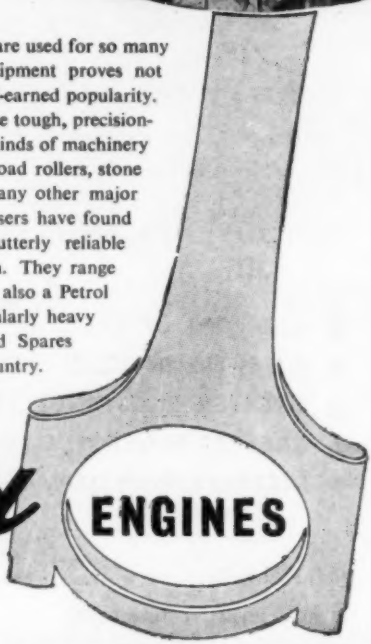

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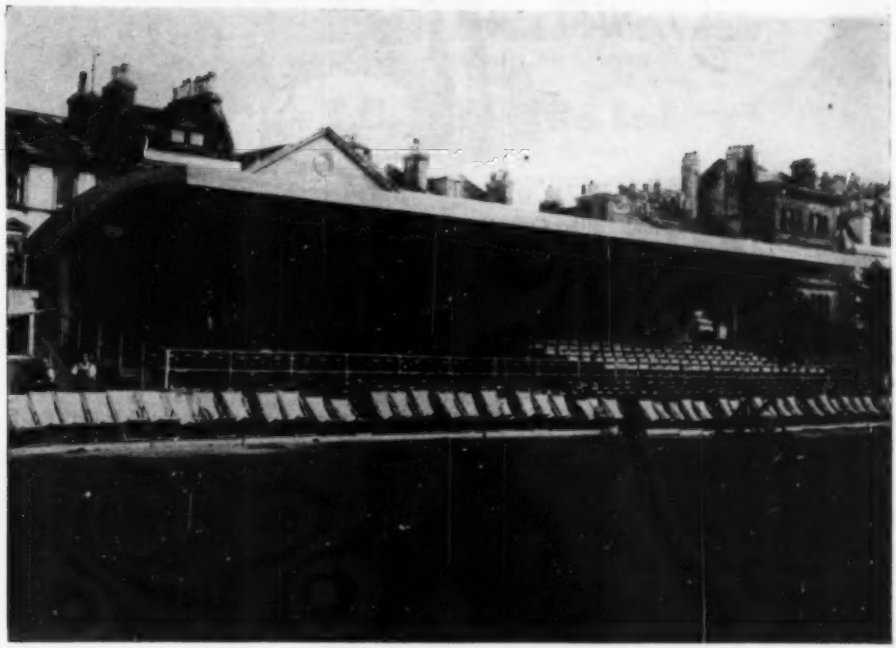

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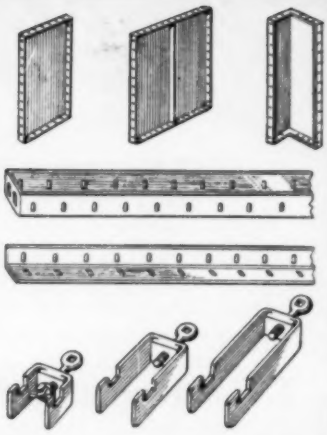

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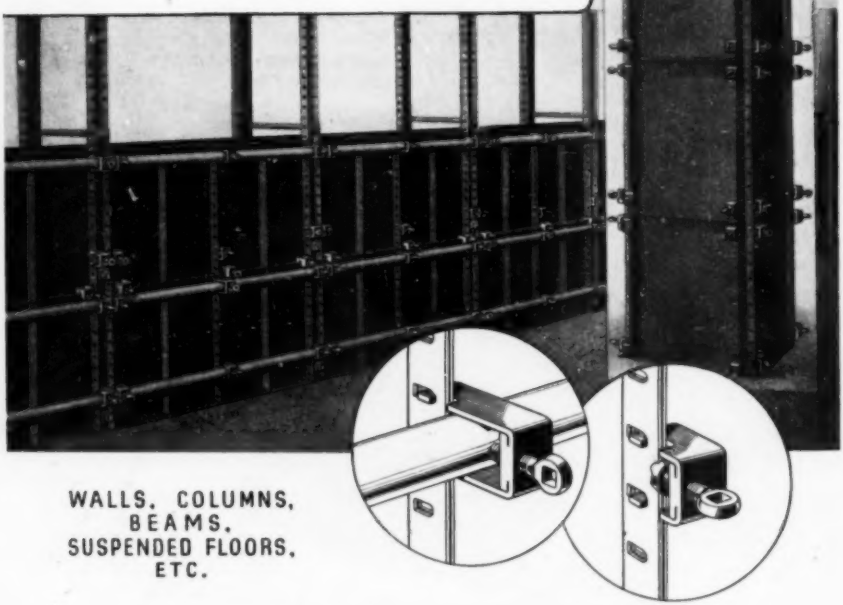

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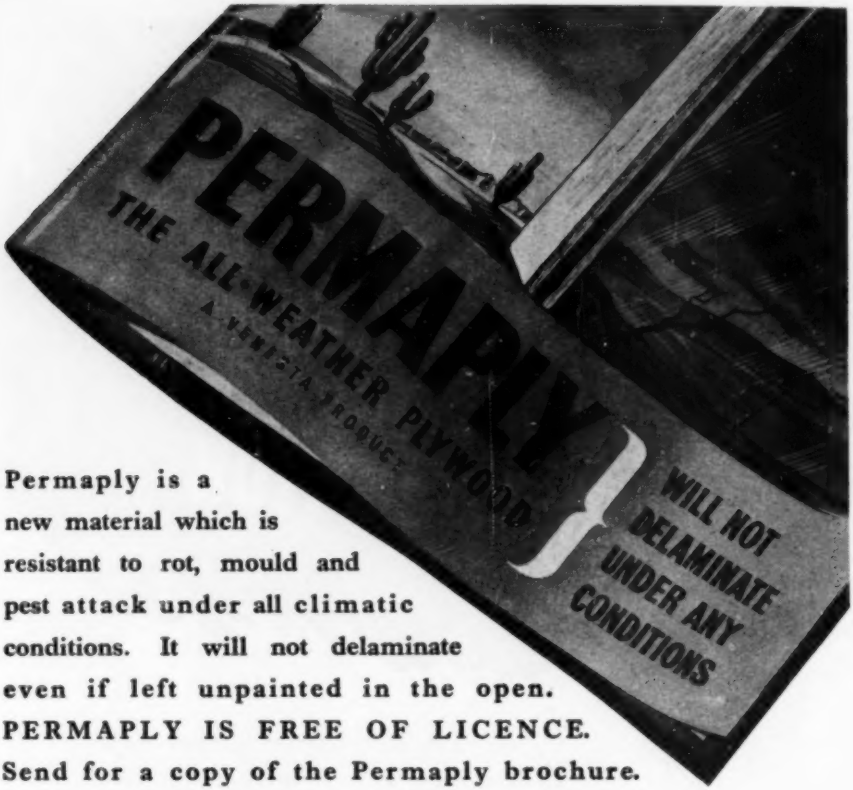

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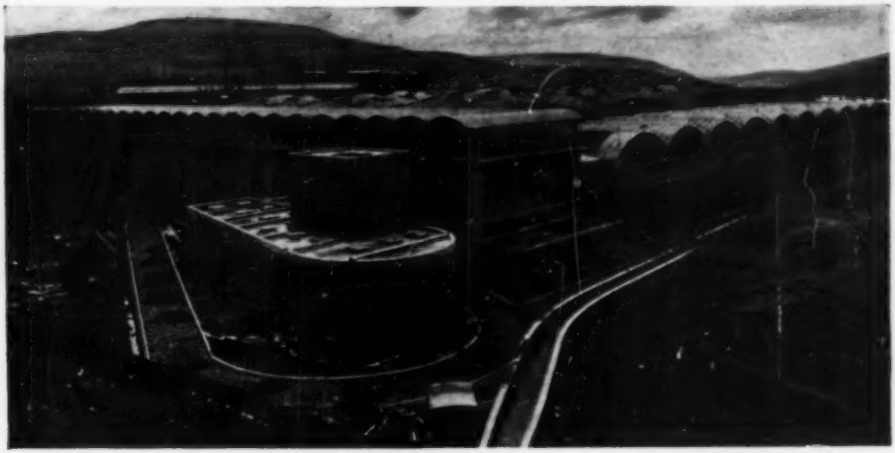

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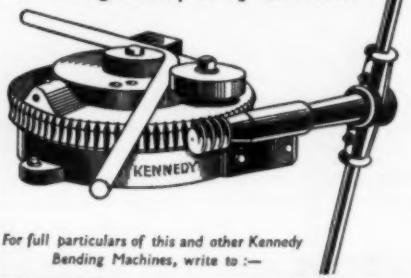

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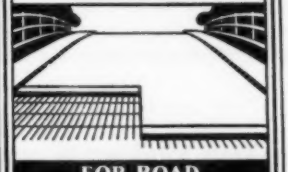

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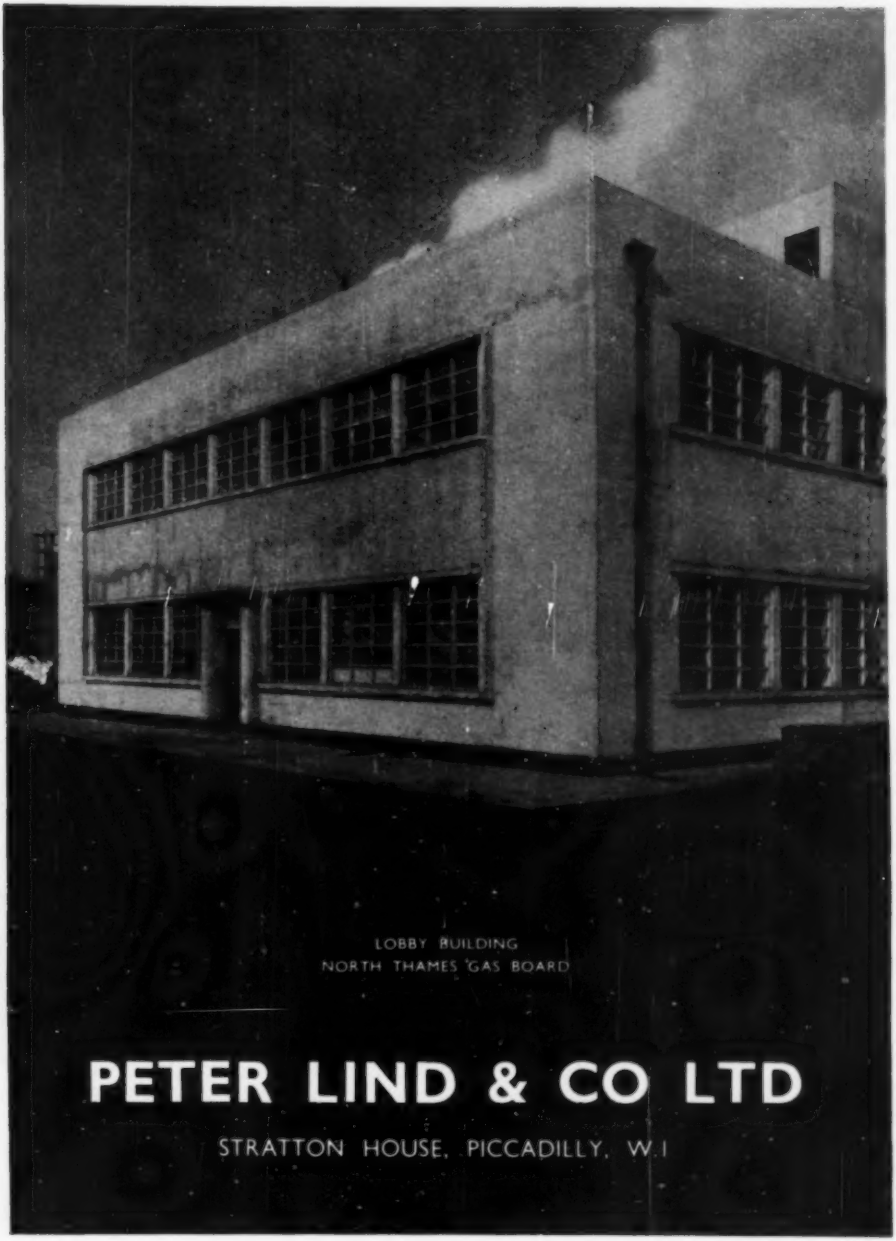
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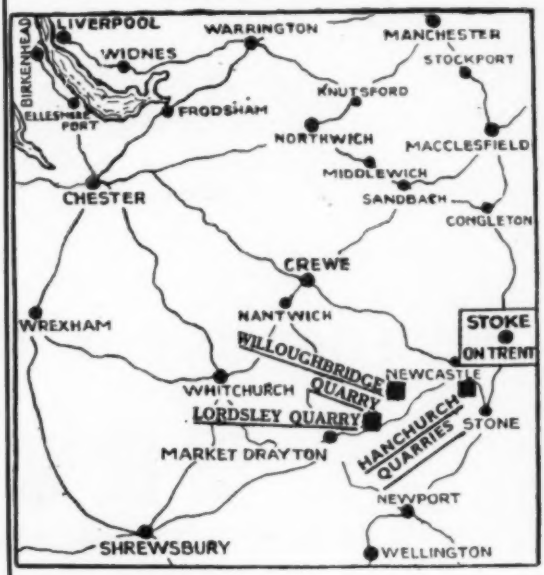

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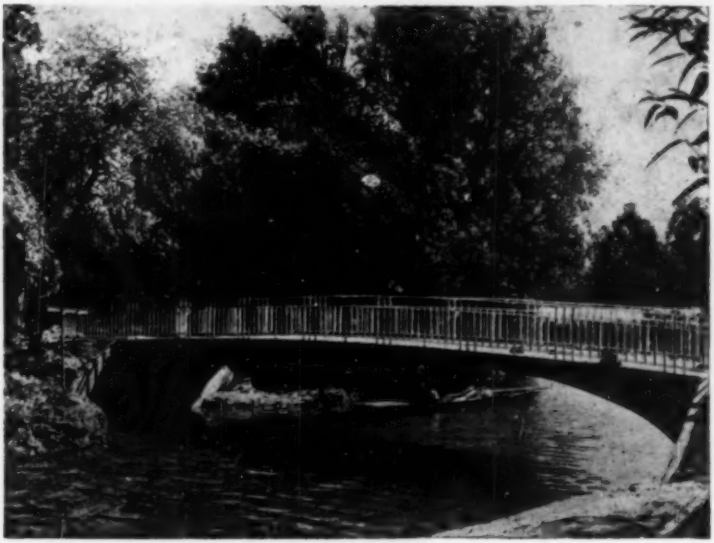

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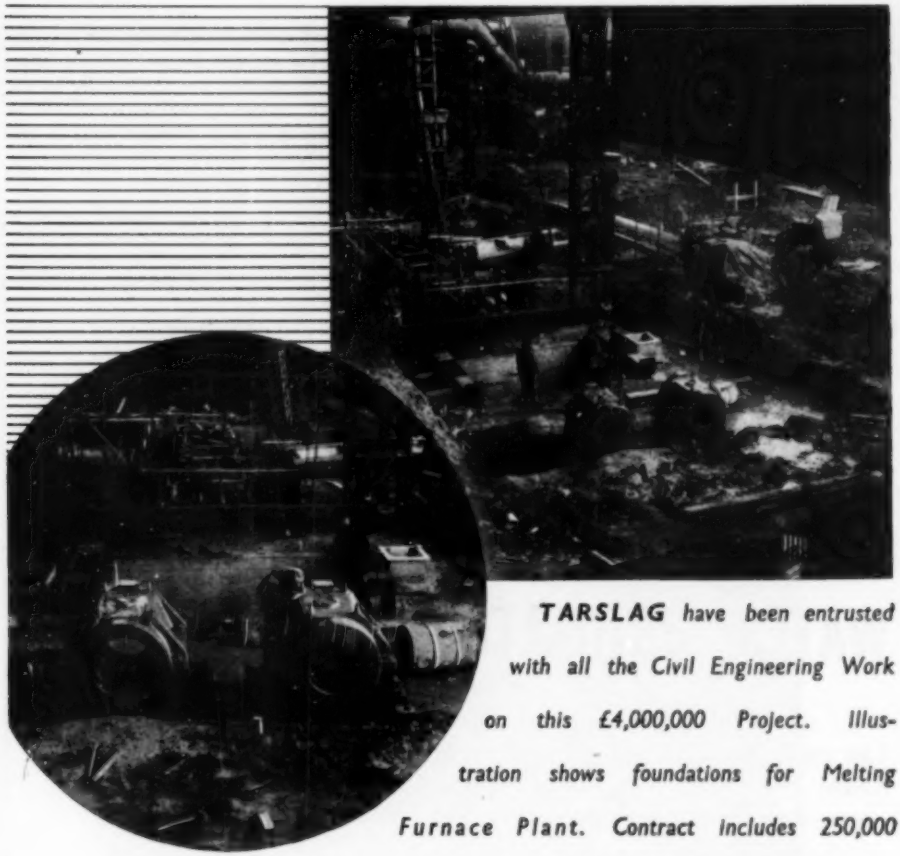

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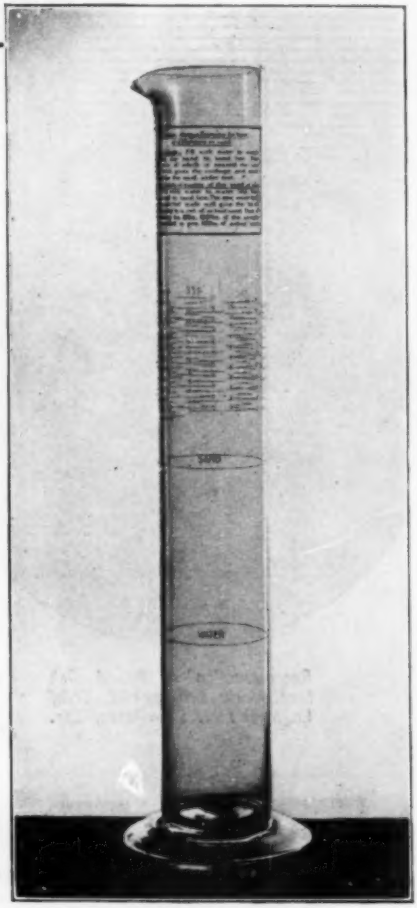

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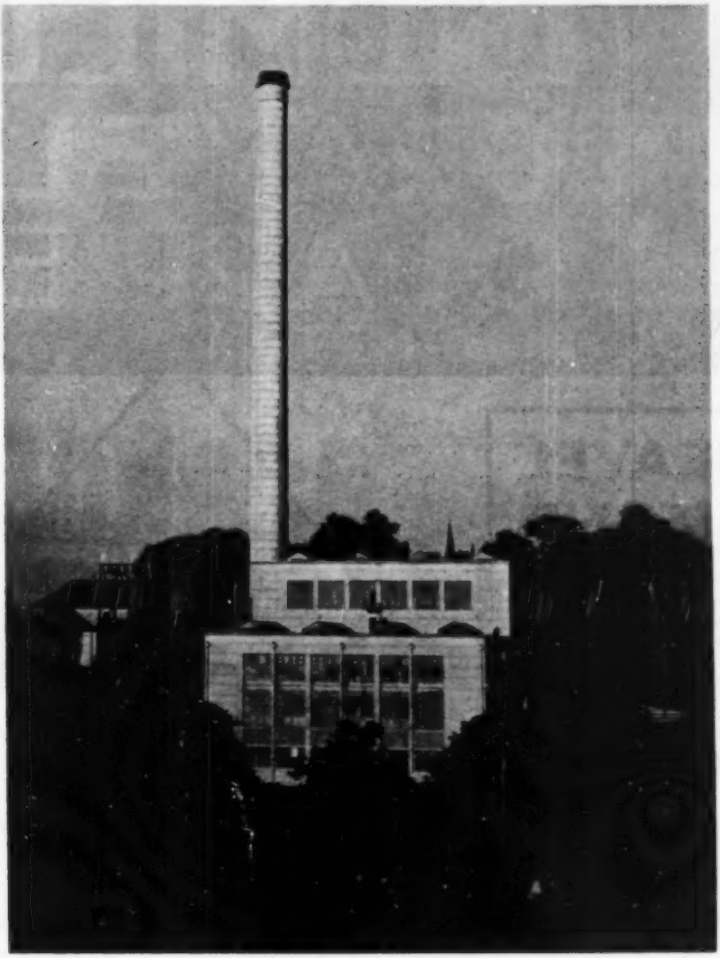

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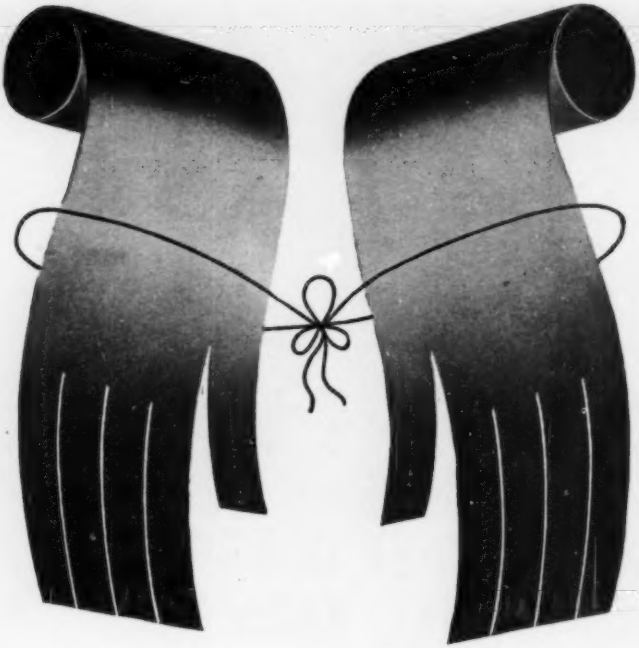

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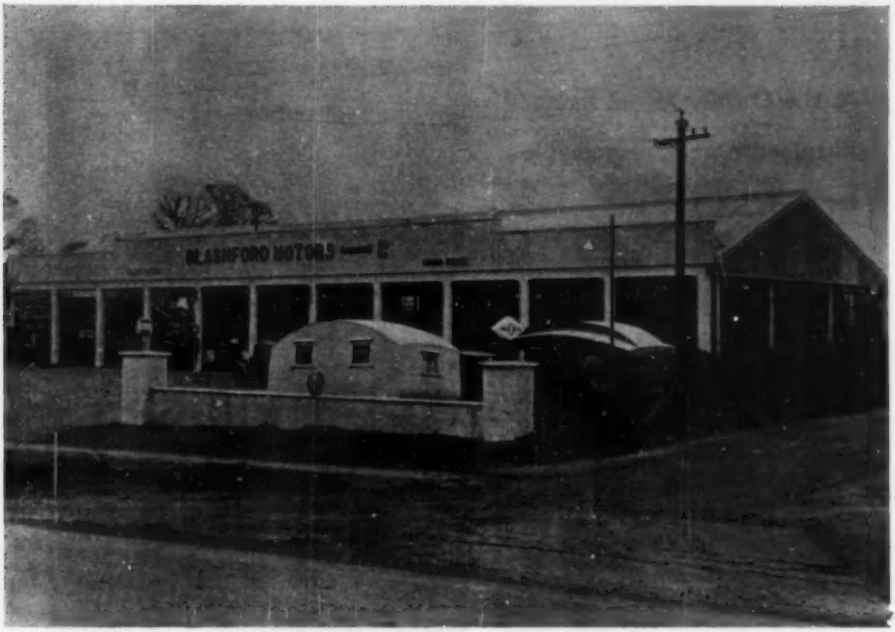

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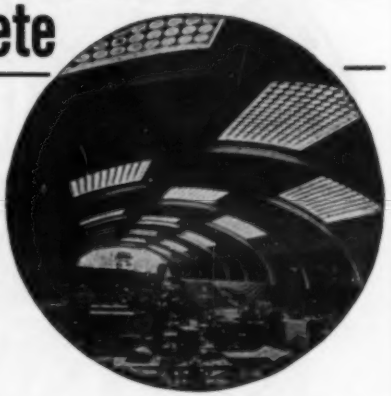

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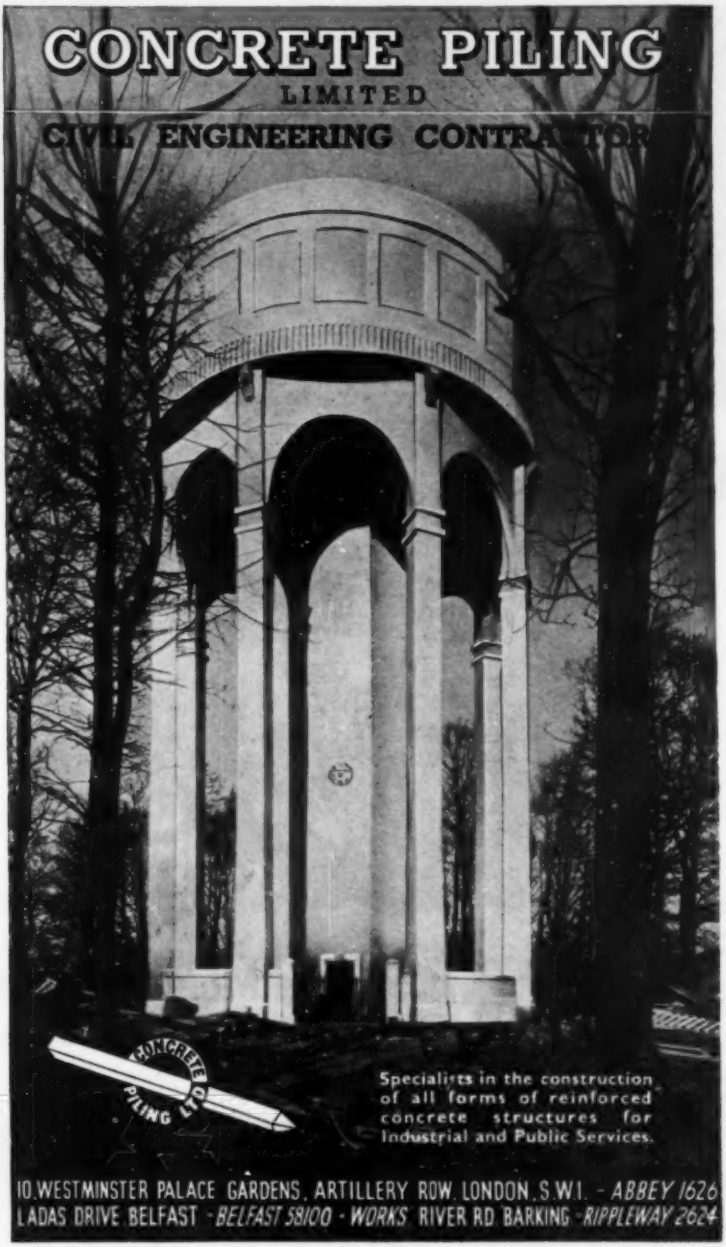
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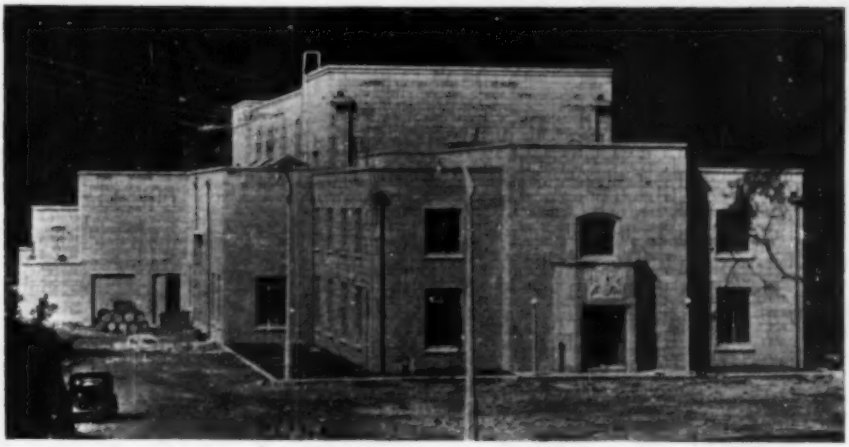

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Volume XLVI. No. 1. LONDON, JANUARY, 1951

EDITORIAL NOTES

The "Trades" of Architecture and Engineering.

ATTEMPTS to find the truth of a statement or the meaning of a word by studying statements or words with opposite or contrary meanings have been popular since Plato and Socrates so skilfully proved that apparently sensible and truthful statements could have no meaning at all or meant something quite different from what those who made them thought that they meant. Some United States judges seem to be similarly gifted. A dictionary says that a profession is a vocation and especially one concerned with a branch of learning or science, and that a trade is a mechanical or mercantile employment which is the opposite of a profession. Yet a United States judge has ruled that doctors are following a trade when they are ministering to the sick, the United States Supreme Court has decided that estate agents are engaged in trade, and the United States Department of Justice has assumed that these decisions may be used as the basis of action against organisations of architects and engineers. Legal action is not proposed, however, merely because the members of these organisations have been found to be tradesmen, or against all such organisations, but only in cases where the organisation has a scale of fees below which its members may not give their services. The attempt to enforce a minimum scale of fees has been found to be an offence against the Sherman Act which prohibits the fixing of prices in restraint of trade. The Supreme Court has ruled that "the fact that a business involves the sale of personal services rather than commodities does not take it out of the category of trade within the meaning of the Act, for the Act was aimed at combinations organised and directed to control the market by suppression of competition in the marketing of goods and services."

The decision that doctors, architects, engineers, or people engaged in other occupations generally called professions, are tradesmen is not therefore based on whether they sell advice, medicine, or drawings or other documents prepared in a home or office, but only on whether or not they have banded themselves together in an association which has issued a scale of charges to which they agree to adhere. This, of course, is what every member of a trade union does when he agrees not to sell his services at a price less than that laid down from time to time by the trade union. Professional societies, however, seldom wish to become trades unions, although the purpose of some of them is little else than to safeguard the financial interests of their members. Indeed in recent years the words profession and association have actually been incorporated in the title of at least one

trade union, and in other cases the titles of trades unions indicate the fact that the membership comprises people working in a profession. In some professions to-day a person working in a subordinate capacity can be a member of a learned society which is not concerned with fees or rates of pay, of a professional society which has a scale of fees for an employer but no scale of wages for employees, and of a trade union which is concerned solely with the rates of pay of employees and in no way takes an interest in the income of the employer (except perhaps to reduce it) and is not concerned with the advancement of an art or science (although by arranging an occasional lecture it may pretend to do so). He can also be a member of an organisation whose aim is to " raise the status " of the whole profession of engineering, and is therefore presumably concerned with the advancement of knowledge, the fees of the employers, and the wages of the employees.

It is possible that in Great Britain the question of professional fees may be raised in a general consideration of business profits, monopolies, the rising cost of commodities, and rates of wages. If this should happen any claim that the scales of fees issued by professional associations were in restraint of trade or a restrictive practice could be avoided if all of them were issued as recommendations only, as is already the case with the scales of fees agreed by some professional associations. A scale of fees which was recommended only would permit the members to accept lower fees if they so desired, and no doubt some would do so. In recent years we have so often seen things happen in this country which we believed could only happen in other countries that he would be a rash prophet who declared that doctors, architects, engineers, and other professional people in Great Britain would never be described as tradesmen for the purpose of the law, and scales of professional fees described as restrictive and monopolistic practices in restraint of the trades of medicine, architecture, engineering, and other occupations which we now call professions.

Most architects will prefer to describe their profession as an art, and their work as the production of artistic designs. This becomes truer year by year as the progress of the application of science increases the complexity of many types of structures, and makes it more and more difficult for an architect to design a structure without employing engineers and other specialists to undertake nearly all the work except the plans and elevations, and even these may be increasingly influenced by the advice and recommendations of the specialists. It is nowadays quite misleading to say that an architect or a consulting engineer is engaged in trade, however true it may have been in the case of the architect when he was the master builder who designed his works and supervised their erection, or in the case of an engineer who is also a contractor. The term engineer is now so commonly abused that it is in danger of becoming meaningless. Not only do machine operators and fitters generally style themselves engineers, but men working in garages often call themselves motor engineers, engine-drivers are locomotive engineers, plumbers are sanitation engineers, and so on. The ancient and honourable word mechanic is falling into disuse as a result of the present tendency to use high-sounding titles which are intended to suggest that a man is what he is not. If an Act is passed which deals with the scales of fees issued by professional bodies it would be better to use in the Act words which truly describe the vocations of the people concerned.

Specification for Concrete Work.

By G. P. MANNING, M.Eng., M.Inst.C.E.

THE fundamental problem of all engineering is the same, namely, to produce the best possible result having regard to all the circumstances in which the work will probably be carried out. It is impossible to produce a really good specification for any work unless all these circumstances are carefully studied. The cogent factors always include: (*a*) The quantity and quality of the materials available (it is assumed that the use of steelwork, brickwork, light alloys, etc., has already been considered and that concrete has been decided on); (*b*) The quantity and quality of labour available; (*c*) The quality of the supervision that will be provided; (*d*) The amount and type of plant available; (*e*) The details of the structure. In military work time is often the over-riding factor. In civilian work cost is nearly always important and often the main factor.

MATERIALS.—The production of cement is not part of the civil engineer's work in this country, and acceptance of the British Standard product can scarcely be avoided.

We all know what kind of sand and shingle we should like, and it is tempting to write a specification describing it. But what happens if none of the local pits can supply it? In parts of Britain the only local aggregate is a porous sandstone, and in parts of Eire the only local sand has a fineness modulus of about 1·0. To transport sand or shingle adds about 4*d*. per ton for every mile of distance from the pit. Where local aggregates are poor a great deal of preliminary test-cube making may be necessary before a final choice of mixture is made. The writer has seen a specification calling for a specially-graded sand which none of the many pits in or near the London area could supply.

Proper specification of concrete work always requires a knowledge of local sources of aggregate. Visits to pits are far superior to the study of small samples. At present aggregates are supplied " all-in " or divided into coarse (shingle) and fine (sand). In future we may have four, namely, coarse shingle, fine shingle, coarse sand, and fine sand. This need not necessitate four heaps of aggregate on the site, as they could be screened, gauged, and remixed at the pit.

LABOUR.—Considering the wage he receives, a really conscientious workman is possibly as efficient as any part of the concrete industry but, being human, he has his limitations. Curiously enough, in a country famous for good quality and sound craftsmanship, the English traditional outlook on concrete as a rough mixture to fill foundation trenches dies hard.

It is useless to specify a high standard of workmanship, or the high standard that we hope our efforts will produce in the next generation. All we can ask for is the best standard reasonably now available, and specifications must be written within this limit. A good set of drawings is a great incentive to good workmanship, yet the most severely-worded specification may be accompanied by a poor and obscure set of drawings. If the engineer produces third-rate drawings, he cannot expect first-class workmanship. Bonus systems are apt to produce a bigger output of poorer work.

SUPERVISION.—Contracts may be roughly classified by their value as follows.

Type I.—A £1,000,000 contract will have a resident engineer and agent, both with assistants, giving continuous supervision and making regular tests.

Type II.—A £100,000 contract will have a clerk of works and a senior general foreman, both of whom, having numerous other duties, can give only intermittent technical supervision to the concrete. Regular visits from the offices of the engineer and contractor are usual.

Type III.—A £10,000 contract will have a foreman, and occasional visits from head office.

Type IV.—A £1000 contract is left to a senior leading hand, with a visit or two if convenient.

It is useless to issue a long technical specification if there will be no one on the site properly to interpret or enforce it. A specification that would be excellent on a large contract would be misunderstood or ignored on a small contract. The length and phraseology must be adjusted to the type of supervision that may be expected.

PLANT.—This is discussed under separate headings in the following.

The Concrete "Chain."

Fig. 1 shows the concrete "chain" of twelve links.

LINKS 1, 2, AND 3. MATERIALS.—These have already been discussed.

LINK 4. WATER.—The necessity for clean mixing water is too well known to warrant further discussion.

LINK 5. HANDLING MATERIALS.—A good specification will forbid the contractor to transport, off-load, chute, or stack sand and shingle in such a way that it will segregate or get contaminated.

LINK 6. PROPORTIONS.—Specification for this link depends on the size of the contract. The cost of transporting, erecting, and dismantling a batching plant, including the pit, and feeding crane, may be £200 in addition to the cost of operation, depreciation or hire charges. If a contract contained only 100 cu. yd.

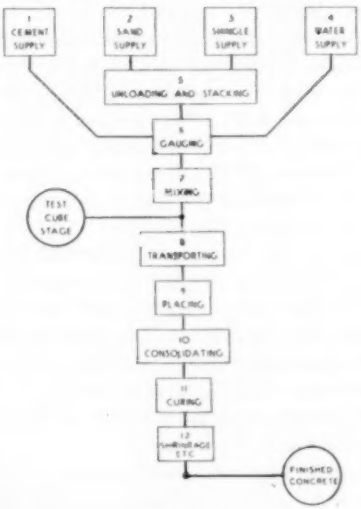

Fig. 1.

the plant alone would cost 40s. per cubic yard in addition to operation and depreciation charges. If a batching plant saves 4s. per cubic yard compared with hand gauging, the contract must contain about 1000 cu. yd. before the plant becomes economical. Mechanical batching can be reasonably specified only on contracts of Types I or II. On a contract of Type IV a visit to the site and careful measurement of the gauging barrows are necessary. A richer mixture must also be specified to allow for errors in gauging on what is virtually an unsupervised contract. Gauging by shovels is still all too prevalent on small works.

When we speak of water-cement ratio we should mean the ratio of water to cement in the finished concrete. The water in concrete may be contained in the shingle, contained in the sand, added at the mixer, rain, or water seeping up from wet ground or wet concrete below. Water may leave concrete by evaporation, leakage through joints in the shuttering, or seepage into dry ground or dry concrete adjoining. It is difficult to write a specification that will take care of the hour-to-hour variations in these eight factors. The slump test is not only inexact and subject to variation with the skill of the operator, but even if it were exact it could take care of only three of these eight factors.

Insistence on an experienced mixer-driver and a general description such as " medium-dry " mixture is all that can reasonably be specified for outdoor unvibrated concrete. The process of specifying a very small slump, followed by heated argument and perhaps ingenious subterfuge by the contractor, and recriminations on both sides, has little to commend it. If the concrete is to be vibrated, special precautions must be specified to eliminate the last five factors or poor concrete may result.

The writer is greatly in favour of the French practice of specifying a minimum content of cement per cubic yard. The suggestion that the engineer should merely specify a minimum cube strength and leave all else to the contractor resembles the cost-plus system of contracting—ideal in theory but very open to abuse in practice.

LINK 7. MIXING.—Whenever it is warranted by the size of the work, the mixer should be big enough to take a batch containing a whole bag of cement. With a 1 : 2 : 4 mixture this means $1\frac{1}{4}$ cu. ft. of cement, $2\frac{1}{2}$ cu. ft. of sand, and 5 cu. ft. of shingle, or $8\frac{3}{4}$ cu. ft. of unmixed materials. This rules out the 7/5 mixer. The engineer should fix the position of the construction joints, and the amount of concrete between these joints often determines the minimum size of concrete plant required. A 10/7 mixer may be expected to average twenty batches of $6\frac{1}{2}$ cu. ft., or 130 cu. ft., per hour. If this size machine is used to mix the concrete for a foundation slab measuring 30 ft. by 30 ft. by 10 ft. thick in hot dry weather, the concrete rises only $1\frac{3}{4}$ in. per hour. If placed in layers 6 in. thick, one layer would be placed every $3\frac{1}{2}$ hours and the " monolithic " foundation would comprise twenty precast slabs 6 in. thick laid on top of one another. The specification must insist on sufficient mixing plant to concrete the work as the engineer intends it to be done.

LINK 8. TRANSPORTING.—Methods used for transporting concrete must not allow it to segregate. This may be interpreted in two ways, namely, to suit the means of transport to the concrete, or to suit the mixture to the means of transport. In a technical paper describing a large reinforced concrete structure it was stated categorically that the only proper means of handling the concrete was by

means of a chuting plant. If this were true then a " chutable " mixture had to be specified. A mixture will flow along a chute if the sum of the unmixed volumes of sand and cement is not less than the volume of shingle. A 1 : 2 : 3 or a 1 : 1½ : 2½ mixture will flow along a chute, but a 1 : 2 : 4 or 1 : 1½ : 3 will not do so without segregation or excessive water content.

LINK 9. PLACING.—As explained under Link 7, successive layers of concrete must be placed quickly following one another. Each layer should not exceed 6 in. in depth.

LINK 10. CONSOLIDATING.—This consists of two parts, namely, consolidating the layer of concrete when placed, and " marrying " this layer into the layer below. It is not only necessary to get each layer into place before it has a chance of setting. Every layer must be in place before the layer below can dry out or set.

If a man makes a 6-in. test-cube in five minutes this is at the rate of two-thirds man-hour per cubic foot. To give the same degree of consolidation to the output of a 10/7 mixer would require 87 men on simple easily-accessible work, or, say, 150 men on a deep, narrow, heavily-reinforced wall. The common rule of " as many men behind the mixer as in front " would provide four men only. The test-cube is therefore a check on the seven links that precede it, but gives no check at all on the five links that follow it. Unless the men can work inside the shutters the distance from the top of the shutter to the top surface of the concrete must be limited (say, 6 ft. maximum) or the men will not be able to see or reach the work. Where mechanical vibration is used, numerous inspection ports must be left. These points must all be dealt with in a good specification.

A frequent cause of poor consolidation is poor shuttering. That is, as the men ram the concrete the shuttering may bulge or shift. The common reaction of striking the bulge with a sledge-hammer liquefies the concrete, increases the lateral pressure, and increases the size of the bulge. Thereafter the concrete would probably be left unrammed. A good specification insists on good shuttering.

LINK 11. CURING.—An important point often overlooked is that timber shuttering is a good insulator. The removal of timber shutters on a cold morning following a day's concreting often causes sudden cooling, with consequent cracking of the green concrete. On the other hand, leaving the shutters in place for several days is a source of delay and difficulty in times of timber shortage. In these times, the engineer must decide in each case which is the lesser evil.

LINK 12. SHRINKAGE.—Dirty materials and excessive mixing water increase the shrinkage of concrete, but even the cleanest and driest concrete will shrink. Keeping the concrete continuously wet will delay the rate of shrinkage, but this is often impossible. Dividing the structure by expansion joints is a matter of design rather than specification, but the effect of shrinkage can often be reduced by insisting on concreting the parts of a structure in a certain order.

Specification and Cost.—The engineer should reckon the cost of every restrictive clause he writes and see if he can obtain the same result by other methods. For example, is it better to use a dry 1 : 2 : 4 mixture and insist on vibration, or to use a 1 : 1½ : 3 hand-rammed mixture ?

Specification and Research.—The writer does not subscribe to the opinion that laboratory research is useless. Unfortunately it is not balanced by adequate outside research, and thus gives one-sided and distorted views.

Specification, Codes of Practice, and Standards.—A code must neces-

sarily be essentially retrospective. A text-book should be up to date and may anticipate the future. Codes and text-books must both range widely in their scope, whereas a specification must be drawn for one particular contract built at one particular time under one particular set of conditions.

Specification and Reinforcement.—Reinforcement should be arranged so that it can be wired together into a strong self-supporting skeleton. If this is not done, either the steel is pushed out of place as the concrete is rammed or the concrete is not properly rammed for fear of displacing the steel.

Specification and Design.—High stresses and small clearances suitable for a large structure could not be obtained or maintained on a small structure, and different standards of design are required for different sizes and types of contract.

Fig. 2.

Specification and Concrete Literature.—Of the three groups associated with the concrete industry—contractors, consulting engineers, and laboratory technicians—by far the most writing is done by the academic fraternity. If concrete is to develop we must have a balanced literature. If contractors cannot be persuaded to become more vocal, then the flood of theory must be checked to preserve the balance.

Some Conclusions.

Fig. 2 shows an interesting example of the use of different water-cement ratios. The two specimens were made of the same materials and in the same way except for the amount of mixing water. That on the left was made of a medium dry mixture (slump about 5 in.) and that on the right of a very dry mixture (slump about 1 in.). These specimens, heavily reinforced, were made vertically and hand-rammed. In these circumstances the wetter mixture is a good one and the drier mixture a very bad one. But if these were small precast products made on a vibrating table the wetter mixture would shake to pieces and the dry mixture would settle into good solid concrete. With a change of conditions the

good mixture becomes a bad one, and vice versa. There is no such thing as a fundamentally good mixture, and no such thing as a good standard specification.

The writer has encountered many opinions on concrete, some expressed as such, some implied in codes of practice or in specifications. These opinions are based on undoubted and undisputed facts, and have been deduced therefrom by intelligent and logical reasoning. Most of them are wrong. Before a jury is asked for a verdict the law insists on three things, namely, (1) the truth, (2) nothing but the truth, and (3) the whole truth. The law realises that an opinion based on anything less than the whole truth may be completely wrong. A specification may comply perfectly with some of the links of the " concrete chain " and some of the conditions on the site, but it will be a bad specification unless it agrees with all the links and all the conditions.

Book Reviews.

" Foundations of Structures." By C. W. Dunham. (London : McGraw-Hill Book Co. 1950. Price 64s.)

THE keynote of this book, which is one of the best of recent publications on the subject, is that planning foundations is more of an art than a science. Although addressed mainly to young engineers the book will serve as a refresher course to those interested in the subject, since it deals not only with simple and combined bases subjected to concentric or eccentric loads, but also with the design and construction of rafts, retaining walls, waterproofing, piles, cofferdams, open and compressed-air caissons, piers and abutments of bridges, and chimney foundations. There are only passing references to foundations for oil-storage tanks. Of special interest is the brief account of present-day views on the resistance of earth to pressures. The author has avoided writing a treatise on the theory of " soils mechanics," and adopts throughout a practical attitude which embraces the broader principles of the strength and settlement of soils based largely on common sense and the limited use of laboratory tests. The many numerical examples are a good feature commonly omitted from studies of " soils mechanics."

The fact that this is an American book, and that most of the examples are in the United States, need cause no difficulty to readers elsewhere, because generally the terms are those used in this country, and a short glossary gives the meanings of some geological and associated terms. The word " foundation " is used variously to mean the ground on which the structure rests or that part of the structure (called the " substructure ") the main purpose of which is to transmit the weight of the structure (or superstructure) to the ground. The diagrams are very clear, and those depicting actual and probable failures are more graphic than the alternative of many words.

" Building Construction and Drawing." Part 1. Elementary Course. By George A. and A. M. Mitchell. [London : B. T. Batsford, Ltd. 1950. Price 10s. 6d.]

THIS is the 20th edition of a book that has been used by students for more than fifty years. It has been again revised and includes many new illustrations.

" The Analysis of Concretes." By S. R. Bowden and E. H. Green. [London : H.M. Stationery Office. 1950. Price 9d.]

AN account of the method used at the Building Research Station for analysing set concrete.

" Calculo de Concreto Armado." By T. van Langendonck. Vol. II. (San Paulo : Portland Cement Association of Brazil. 1950. No price stated.)

THIS volume of more than 600 pages on reinforced concrete design is in the Portuguese language and the numerous examples are in the metric system. It deals only with beams, and rectangular beams in particular, but includes combined stresses and prestressed concrete. The subject is thoroughly discussed, and many matters commonly omitted from books on design are dealt with at length, as, for example, direct force combined with bending about an oblique axis. There are more than 500 diagrams and 24 design tables.

Reinforced Concrete Residential Flats at Edinburgh.

THE Westfield estate at Gorgie, Edinburgh, comprises six eight-story blocks of flats (A to F) constructed in reinforced concrete (*Fig.* 1). The blocks, which contain 88 flats and are each 30 ft. wide, are arranged in plan in the form of a crescent.

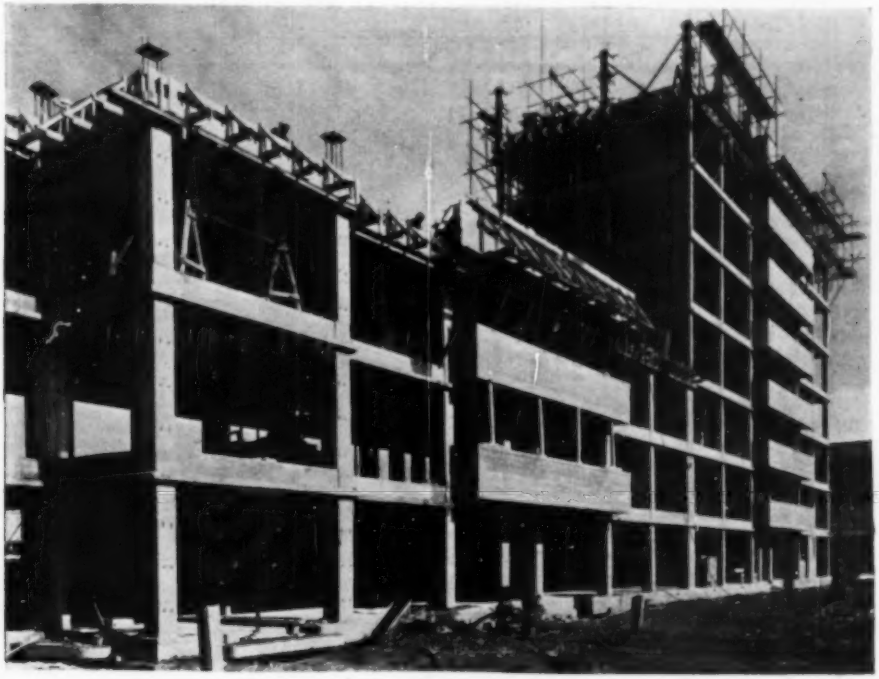

Fig. 1.

Blocks B and C are curved in plan to an inner radius of 150 ft. Block D is 58 ft. long, and the other blocks are about 80 ft. long. The story height is 9 ft. 10 in. in the lowest story, 9 ft. 4 in. in the upper stories, and 11 ft. in the classrooms of the nursery schools in the top stories of blocks A to D. In block E there is a basement 12 ft. deep, 60 ft. long, and 45 ft. wide, containing a heating chamber and fuel store.

The columns, beams, floors, stairs, and balconies are of reinforced concrete (*Fig.* 2). The exterior walls are 10 in. thick and comprise an outer leaf of 2-in. precast concrete slabs, a 3½-in. cavity, and a 4½-in. brick inner leaf. The slabs are erected with continuous vertical joints (*Fig.* 3) and are carried on an exposed projecting nib 4½ in. deep at each floor. The end walls and the walls between the blocks and around the stair-wells and lift-wells are of brick. Partitions are of 3-in. hollow-tile construction. At the expansion and contraction joints (*Fig.* 7), which occur between the blocks except between blocks D and E, the columns and beams are doubled and a 4½-in. brick wall is built in each block with a 3-in. cavity between the walls of adjacent blocks.

The structures are designed to carry the loads specified in the British Standard

Code of Functional Requirements of Buildings, "Chapter V," and the design of the reinforced concrete members is in accordance with the B.S. Code of Practice CP.114 (1948). The reinforcement (*Fig.* 2) in columns is round mild steel bars; elsewhere, including the stirrups in the beams, it is twisted square bars the permissible stress in which is 27,000 lb. per square inch.

The ground at the site, which is nearly level, consists of 11 ft. of loose gravel and 2 ft. of soft silt overlying boulder clay. The buildings are supported on 350 cast-in-situ piles of 16 in. to 18 in. diameter, each pile being reinforced with six $\frac{5}{8}$-in. bars and $\frac{1}{4}$-in. helical binding of 6-in. pitch. The piles are from 17 ft. to 22 ft. long and are driven into the clay in groups

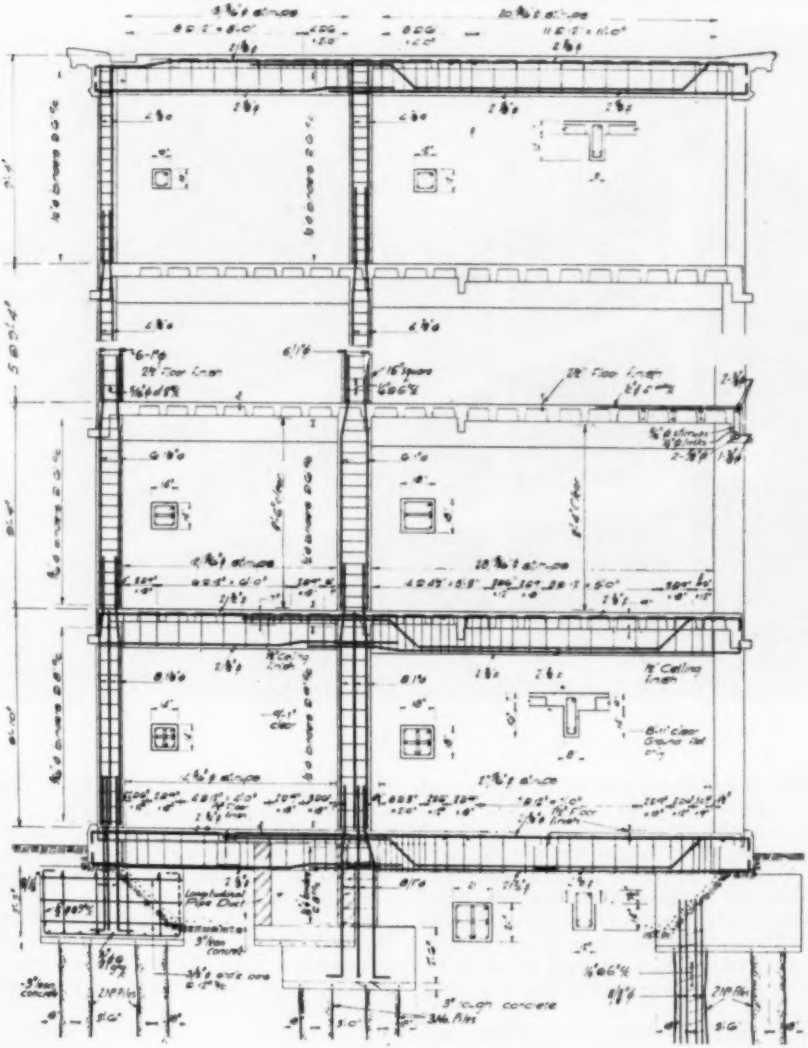

Fig. 2.—Transverse Section.

Fig. 3.—Wall Slabs.

Fig. 4.—Underside of Ribbed Floors.

of two, three, or four under each column. There are no piles under the basement of block E, the weight of this structure being carried on reinforced concrete pad foundations cast integral with the basement floor and bearing on piers of plain concrete which extend not less than 6 ft. into the clay. As the water-table is about 5 ft. below the surface, the basement is waterproofed with a double layer of impervious sheeting weighing 10 lb. per square yard. The sheeting passes between the basement floor and the tops of the piers; between the piers it is laid on 3 in. of lean concrete, and is continued up the outside of the basement walls where it is protected by 4½ in. of brickwork.

The ground floors are constructed with inverted trough-shape beams (*Fig.* 6), 5 ft. to 8 ft. 6 in. long, 16 in. wide, and 7 in. deep, precast at the site. The hollow was formed by No. 10-gauge pressed steel moulds, 11 in. to 12 in. wide and 5 in. deep, having 1-in. lips on each edge, and in lengths of 3 ft. and 6 ft. overlapped to give the length required. The precast beams bear on the pile caps and on rectangular cast-in-situ beams (*Fig.* 5) which span between the pile-caps. The finish of the ground floor is ¾-in. mastic asphalt laid on 1-in. mortar screed in living rooms and 1¼-in. granolithic in kitchens and passages.

The upper floors (*Figs.* 4, 6, and 8) are cast-in-situ ribbed slabs, 7 in. and 9 in. deep, the spaces between the ribs being formed by the steel moulds used for the precast beams. In the straight blocks the ribs are 3½ in. wide at the bottom, and in the curved blocks 5 in. wide, the reinforcement being bent in situ to suit the curve. Hardwood fillets are cast in the bottom

Fig. 5.—Cast-in-Situ Beams of Ground Floor.

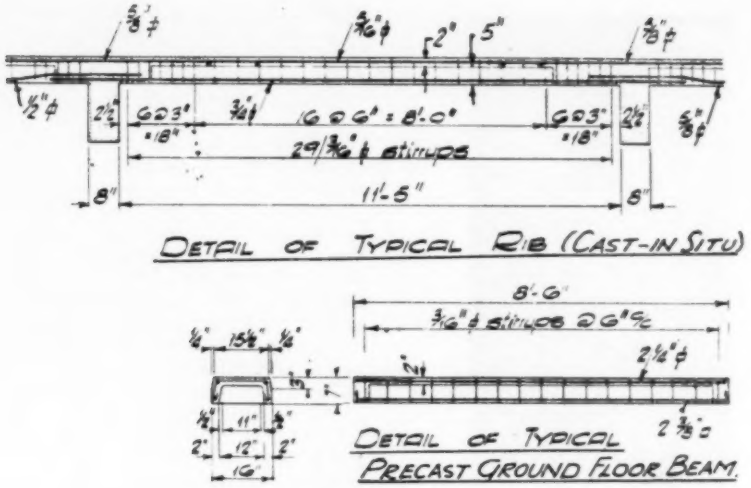

Fig. 6.—Details of Floors.

of the ribs (*Fig.* 4) for the attachment of ceilings. The finish to the upper floors is $\frac{7}{8}$-in. timber boards on runners in living rooms and classrooms, and 2½-in. granolithic in kitchens and passages. The playground on the roof is finished with granolithic. The roofs are covered with 2 in. to 4 in. of breeze concrete on which is bituminous sheeting.

Details of the expansion joints between the structures are given in *Fig.* 7. The 3-in. gap between the roof beams of adjacent structures is covered by a 16-gauge zinc plate (*D*) 9 in. wide; this is secured to the structural concrete on one side by $\frac{3}{4}$-in. sherardised rag-bolts and is free to slide on the other side. There is a $\frac{3}{4}$-in. gap in the breeze concrete (*B*) and the asphalt (*A*), in which is inserted a strip of bituminous expansion-joint filler (*C*). The 1-in. gap between the double columns is closed by a strip of impervious sheeting (*F*) similar to that used in the basement. The joint is continued as a rebated joint through the wall slabs and is sealed with a plastic filler (*E*).

Water-pipes, drains, and flues from the incinerators (one of which is provided in

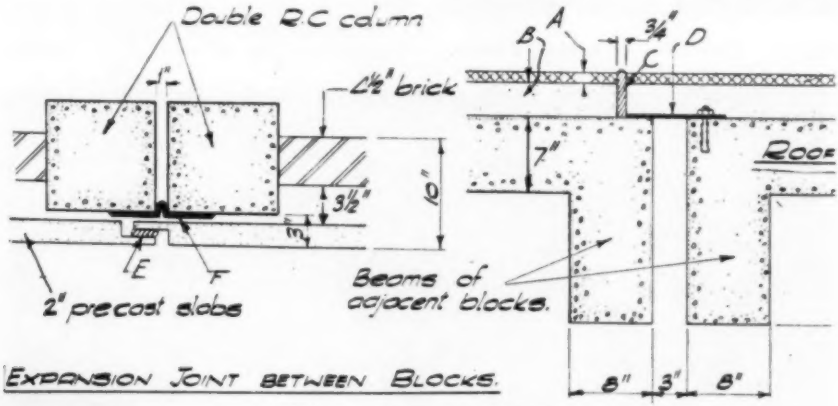

Fig. 7.—Expansion Joints.

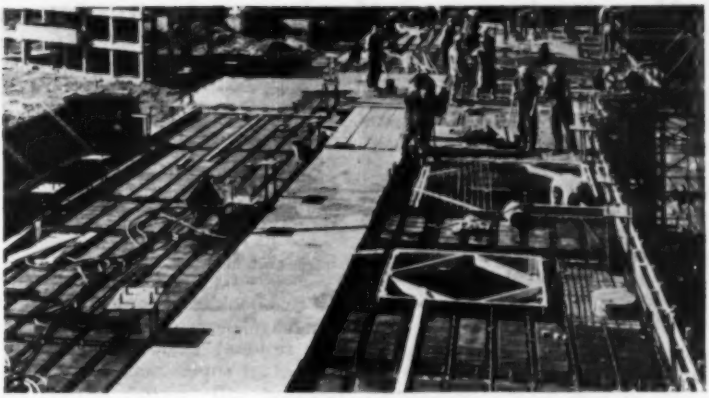

Fig. 8.—Construction of Upper Floor.

each flat) are contained in two vertical ducts in each block. The sides of the projecting part of the ducts are of reinforced concrete and are used as columns.

The mixture of the concrete is about 1 : 2 : 3 by volume, the materials being washed sand, crushed whinstone, and ordinary Portland cement. A weigh-batching machine (*Fig.* 9) is used and is loaded by a ¾-cu. yd. mechanical shovel. The weights of the materials in each batch are corrected to allow for the moisture in the aggregates and for particles of sand larger than $\frac{1}{16}$ in. The concrete is mixed in two mixers and is discharged into petrol-driven wheelbarrows, which are lifted to the upper floors by hoists.

Work commenced on the site in July, 1949, piling commenced in September and was completed by December, 1949. It is expected that the first block of flats will be occupied sometime this year. The buildings were designed for the Edinburgh Corporation by Messrs. Williamson & Hubbard, F./A.R.I.B.A., the consulting structural engineers being Messrs. Kinnear & Gordon. The contractors are Messrs. Hepburn Brothers, Ltd. The wall slabs are made by the Scottish Orlit Co.,, Ltd., and the piles were driven by the Franki Compressed Pile Co., Ltd.

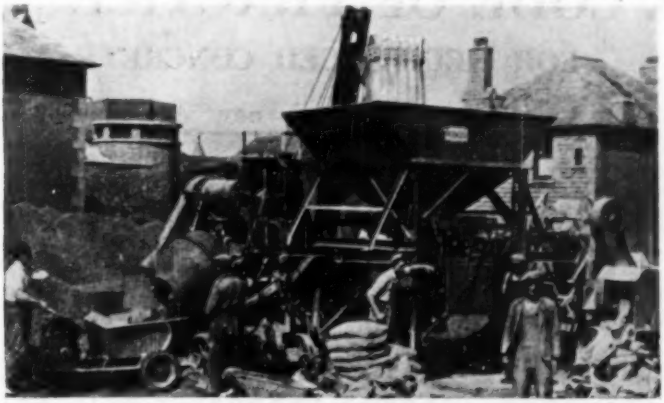

Fig. 9.—Concrete Mixing Plant.

Recent German Books on Concrete.

"Statik Leichtverständlich Dargestellt." By Z. Stiegler. Part IV. (Berlin: Wilhelm Ernst & Sohn. 1950. Price 3.60 DM.)

THIS addition to a series of small books, in the German language, deals briefly with some common problems in the design of structures, including moving loads and influence lines, bending of irregular sections, jointed and continuous beams, three-pin arches, two-pin rigid frames, bending combined with compression of reinforced concrete members, horizontal shearing stresses in timber beams, and plate girders.

"Bemessungsverfahren." By E. Loser. 12th edition. (Berlin: Wilhelm Ernst & Sohn. 1950. Price 12 DM.)

THE twelfth edition of this well-known book, in the German language, differs from the previous edition only by some small alterations and additions to comply with the latest German regulations. The design table originally based on a tensile stress of 21,350 lb. per square inch (1500 kg. per square centimetre) is replaced by one based on 22,750 lb. per square inch (1600 kg. per square centimetre) to conform to the new regulations for bridges.

"Die Zweiseitig gelagerte Platte." By H. Olsen and F. Reinitzhuber. (Berlin: Wilhelm Ernst & Sohn. 1950. Price 16 DM.)

FROM the differential equation of the curvature of a planar slab supported along two edges, formulæ for bending moments and deflections for slabs of any ratio of length of sides and for any loading are derived by consideration of influence areas. Numerous tables and diagrams enable the results to be applied to design and provide means of determining the amount of main reinforcement and distribution bars in slabs spanning in one direction.

"Lehrbuch für Betonbauer." By H. Rietli. (Berlin: Wilhelm Ernst & Sohn. 1950. Price 4.80 DM.)

IN this booklet of 89 pages, in the German language, concrete practice is described for resident engineers, clerks of works, and other supervisors. The composition, making and placing, curing, properties, and behaviour of concrete under various external influences are briefly described. The illustrations include several photographs of appliances used in Germany for vibrating concrete.

"Der Stahlhochbau." By B. C. Kersten. 5th edition. (Berlin: Wilhelm Ernst & Sohn. 1949. Price 15.60 DM.)

THIS is an introduction to the practical design of steel buildings. Fabrication and workshop practice are dealt with, and consideration is given to the design of riveted connections and welding. The purpose of the book is to give sufficient practical information to enable a student to work usefully in a design office, intricate calculations being avoided.

B.S. CODE OF PRACTICE (CP. 114–1948)

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Concrete Construction in the Tyne Tunnels.

"SHELL" DOMES OVER ENTRANCES.

THE two tunnels under the river Tyne between Jarrow on the south bank and Howdon on the north bank, which were commenced in 1947, are now nearing completion and comprise a tunnel of 10 ft. 6 in. internal diameter for pedestrians and one of 12 ft. diameter for cyclists. The scheme includes a tunnel for vehicles, the construction of which is postponed. The two tunnels are parallel under the river but are connected at each end and at one intermediate point. Access is by a lift

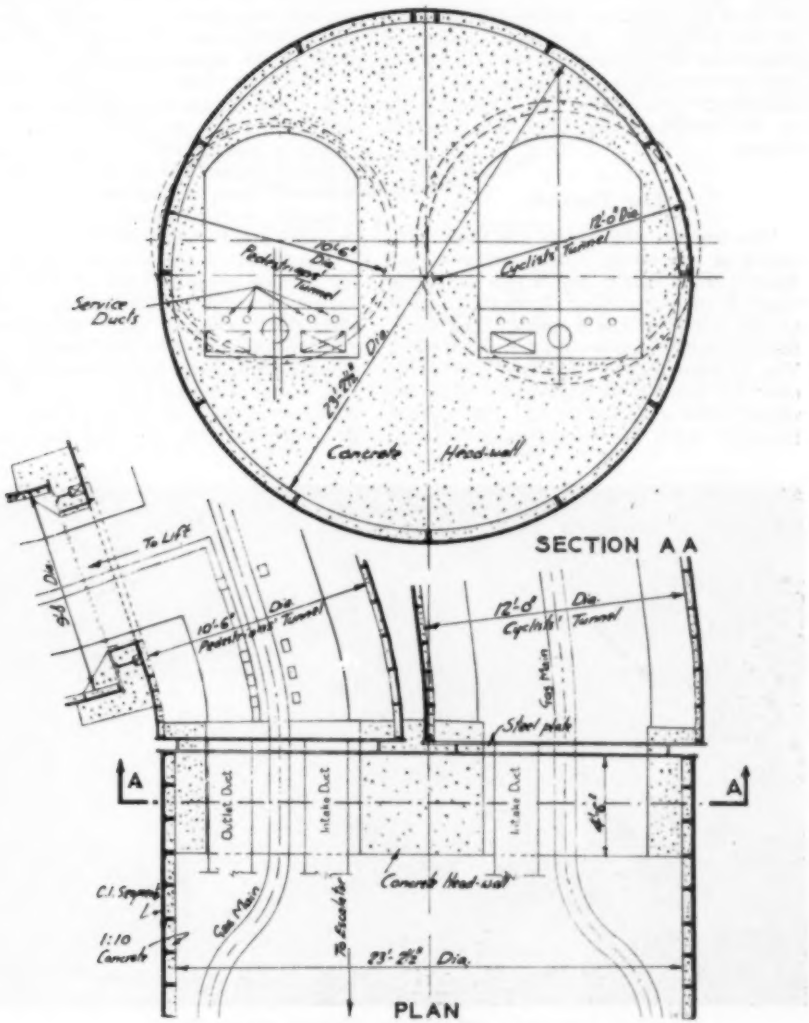

Fig. 1.—Concrete Head-wall.

giving direct access to the pedestrian tunnel and by escalators at each bank. The length of each tunnel between the bifurcations is about 890 ft., in addition to which there are about 190 ft. of horizontal and inclined tunnels for the escalators. The lift shafts are 15 ft. diameter and about 85 ft. deep, and are used as working shafts. The escalators, which are thought to be the longest in the world, are 170 ft. long and are contained in tunnels 18 ft. 6 in. diameter inclined at 30 deg. to the horizontal. The short length of horizontal tunnel between the bifurcation and the foot of the escalators is 23 ft. 6 in. diameter. The roofs of the entrance halls to the escalators are reinforced concrete domes.

The Tunnels.

The tunnels and shafts are lined with cast-iron segments filled between the flanges with fairly dry 1 : 10 concrete, which is placed without shuttering except at the crown of the tunnels. The inner faces of the tunnels are rendered and tiled. The greatest depth of water and earth over the tunnels at high-water ordinary spring tides is about 65 ft. The ground through which the tunnels are bored is variable, and towards the north bank comprises mainly water-logged coarse shingle in the lower part of the bores and silt and similar soils in the upper part. About two-thirds of the length of the tunnels from the north bank were therefore driven with a shield in compressed air at 40 lb. per square inch ; compressed air was in use for about twenty months. Towards the south bank a stratum of clay above the bore provided a seal and in this part the tunnels were driven in free air. Driving proceeded for the full size of the bore from both banks simultaneously.

The head walls at the bifurcations, where the tunnels of 10 ft. 6 in. and 12 ft. diameter meet the 23-ft. 6-in. tunnel (*Figs.* 1 and 2), are of plain concrete 4 ft. 6 in. thick, and are faced on the river side with 1-in. welded steel plates. The concrete walls provide the structural stability and the steel plates form the seal.

The floors of the tunnels are designed for a superimposed load of 100 lb. per square foot and are constructed of precast reinforced concrete slabs supported at their ends on concrete benching projecting from the lining and intermediately on precast concrete beams and posts. The construction of the floor of the cyclists' tunnel is shown in *Fig.* 3 ; the floor in the

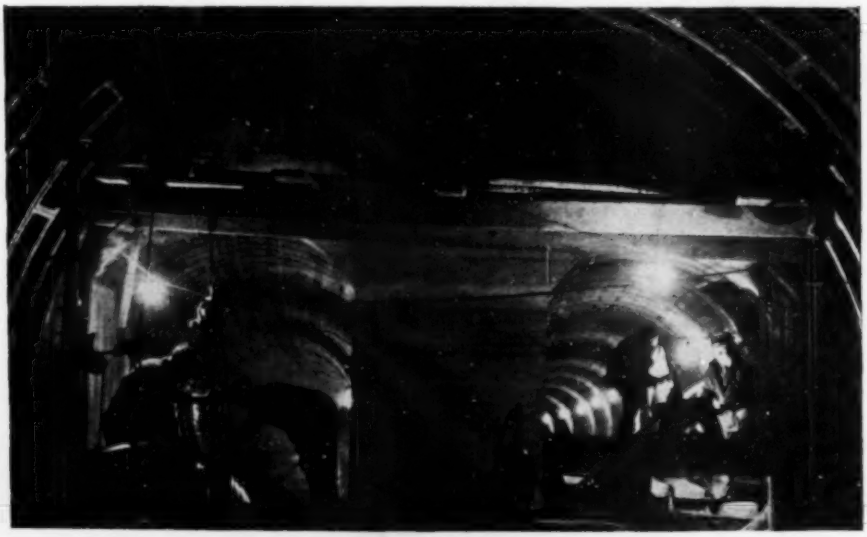

Fig. 2.—Head-wall at Change of Section.

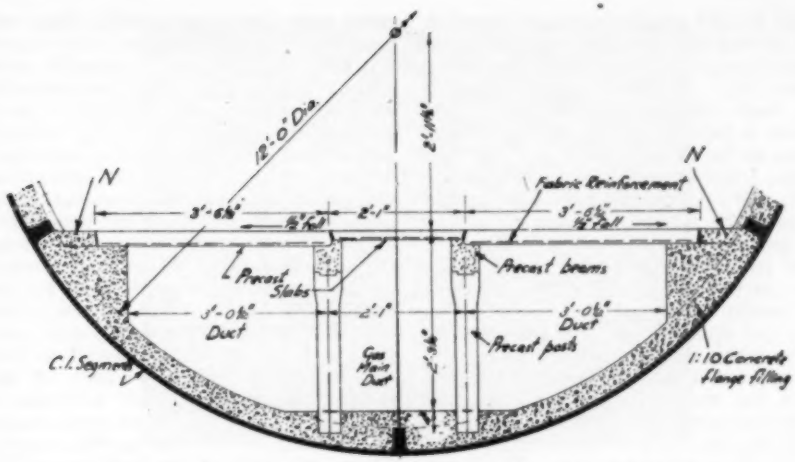

Fig. 3.—Cross Section of Floor of Cyclists' Tunnel.

pedestrians' tunnel is similar but is about 1 ft. 6 in. less in width. Below the central bay of the floor there is a gas main. The slabs over the side bays, the spaces below which are ventilation ducts, are 2½ in. thick and generally about 1 ft. 8 in. wide, and are reinforced with hard drawn wire fabric placed ½ in. from the bottom of the slab and weighing 2¾ lb. per square yard in the cyclists' tunnel and 1⅞ lb. per square yard in the pedestrians' tunnel. The mixture of the concrete is 1 : 1½ : 3; the aggregates are generally ⅜-in. whinstone and sand complying with the requirements of grade A of B.S. No. 882. The ½-in. topping contains ¼-in. crushed granite. The parts *N* (*Fig.* 3) of the concrete benchings are made up to the level of the floor with granolithic, with a carborundum finish, after the slabs are in position and the mortar joints have set. There are about 1000 slabs over the side bays in each tunnel. Steel bed-plates with holes for lifting-keys are cast in some of the slabs. The slabs over the central bay are similar but are 1¼ in. thick.

The Domes.

The domes over the two entrance halls to the escalators are concrete shell structures almost elliptical in plan, with a major axis of 60 ft. 6 in. and a minor axis of 54 ft. 6 in. between the inner faces of the continuous precast concrete lintel (*A* in *Fig.* 4) on which the roof is supported. The lintel is carried on a brick wall (*Fig.* 5) beyond which the roof cantilevers about 5 ft.; the canopy so formed is concentric with the dome and provides protection for the windows. A layer of bituminous felt (*G*) between the lintel and the roof enables the dome to move without restraint and without imposing horizontal thrusts on the lintel. The rise of the dome is 8 ft. and the radius in the vertical plane is 60 ft. The ratio of the clear span to the rise is 7·5. The thickness of the concrete shell is 2½ in. except for a distance of 9 ft. from the lintel where it increases gradually until at the lintel it is 9 in. The proportions of the concrete are 1 : 1½ : 3 with ⅜-in. coarse aggregate. The domes and cantilevers are designed for a uniformly-distributed load of 30 lb. per square foot, which allows for the effects of wind, snow, and other superimposed loads, in addition to the weight of the dome. The dead weight is about half of the total design load.

The reinforcement (*Fig.* 4) comprises principally a tension ring of mild steel bars (*D*) over the lintel and hard drawn welded steel wire fabric (*E* and *K*) in the shell and cantilever. The four 1-in. bars (*D*) are in long lengths bent to a radius of about 30 ft., and at the splices, which are staggered, there is an overlap of 3 ft. 6 in. The bars, the design stress in which is

18,000 lb. per square inch, are bound at 9 in. centres by ½-in. links the upper parts of which project through the meshes of the fabric (*E*). Reinforcement comprising ¾-in. bars (*H*) placed radially at 2 ft. centres is provided in the bottom of the domes at the springing, and is laced with several ¾-in. bars bent to a radius of about 25 ft. and overlapping 1 ft. 9 in. at the splices. The weight of the fabric (*K*) in the greater part of the dome is about 4¾ lb. per square yard; in the top of the lower part of the dome and the cantilever the weight of the fabric (*E*) is about 9½ lb. per square yard. The tensile stress in the fabric does not exceed 25,000 lb. per square inch, and the cover of concrete is not less than ¾ in.

The centering was constructed of steel tubes (*Fig.* 6). The circumferential tubes were in short straight lengths. The radial tubes were curved in a vertical plane and provided a support at about 18-in. centres for light metal ribs of inverted-V cross section (*B* in *Fig.* 4), which are generally at 4-in. centres and to which is wired small-mesh expanded metal (*Fig.* 7) which acts as reinforcement resisting secondary bending. After placing the reinforcement (*Fig.* 8) the concrete was deposited on the expanded metal in concentric bands each about 3 ft. wide commencing at the springing and proceeding towards the crown. Timber shuttering was provided for the cantilevered canopy (*Fig.* 9), which was not concreted until the domes were complete and the centering removed, so that the slight outward movement of the springing of the dome due to stress in the tension band due to the dead weight did not impose a strain on the canopy. The rough soffit of the slab formed by

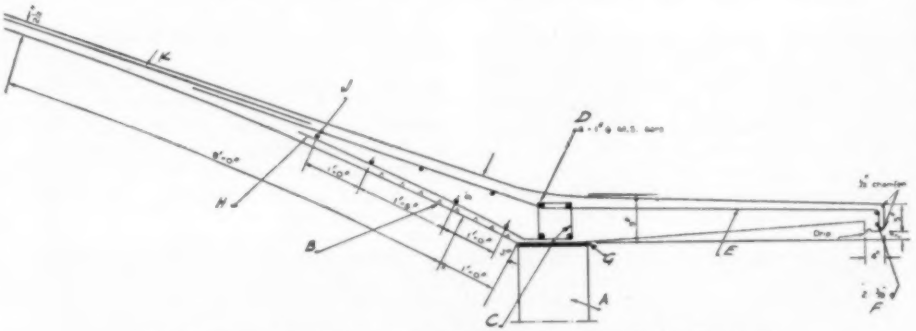

Fig. 4.—Part Cross Section of Dome over Escalator Entrance.

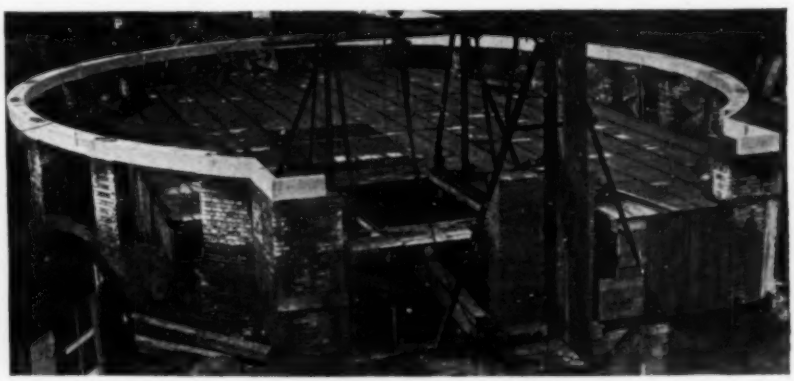

Fig. 5.—Lintel to carry Dome over Entrance Hall.

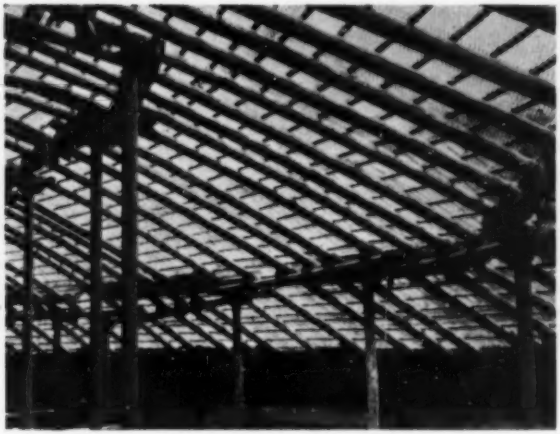

Fig. 6.—Tubular Steel Centering and Expanded Metal in Position for Dome.

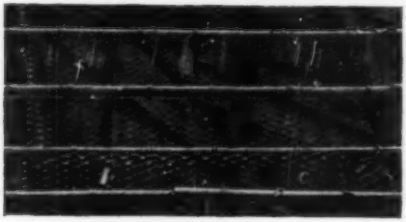

Fig. 7.—Expanded Metal and Ribs.

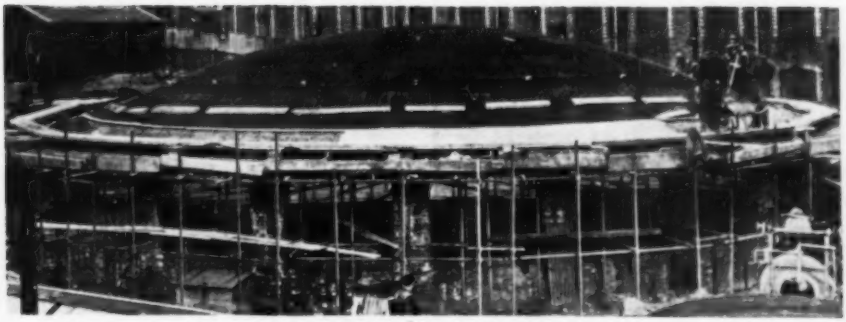

Fig. 8.—Dome in Course of Construction.

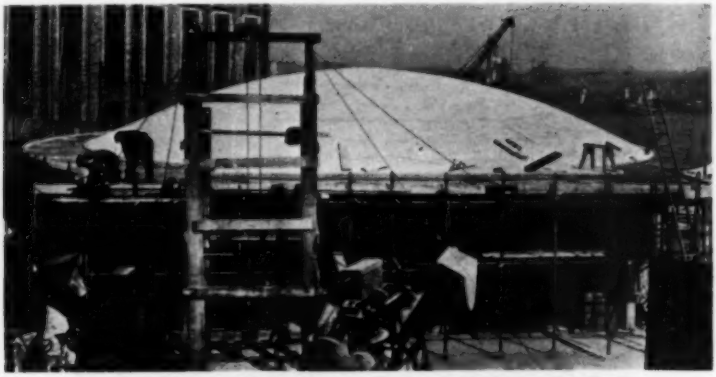

Fig. 9.—" Shell " Dome over Entrance Hall.

depositing the concrete on the expanded metal acts as a key for 1 in. of asbestos-cement, which was sprayed on the underside of the dome. The top of the dome and canopy is covered with bituminous felt treated with aluminium.

Domed roofs were preferred because of the uninterrupted space provided around the tops of the escalators, and because of the large saving in steel. The consulting engineers are Messrs. Mott, Hay & Anderson, and the contractors Messrs. Charles Brand & Son, Ltd. The domes were designed by Mr. C. V. Blumfield, and the sub-contractors for their construction were Messrs. Purdie, Lumsden & Co., Ltd.

Bound Volumes of " Concrete and Constructional Engineering "

BINDING cases for annual volumes of " Concrete and Constructional Engineering " can be supplied in cloth-covered boards lettered in gold on the spine with the title, volume number, and year of publication. Copies for binding should be sent post paid to Concrete Publications, Ltd., 14 Dartmouth Street, London, S.W.1. Where possible, missing numbers will be supplied at the published price to make up incomplete sets, but as many of the numbers published during the past few years are not available it is advisable to ask the publishers whether they have the numbers required before sending incomplete sets. The cost of cloth-covered lettered cases is 4*s.* 9*d.* for each volume. The cost of supplying a case and binding a volume is 10*s.* 10*d.*, including packing and carriage.

Congress on Prestressed Concrete.

As part of the celebration of the 75th anniversary of its foundation, the Association des Ingénieurs sortis des Ecoles Spéciales de Gand (A.I.G.) is holding an international congress on prestressed concrete at Ghent from September 10 to 13, 1951. The president of the organising committee is Professor G. Magnel. The congress will discuss all systems of prestressed concrete, and a special tribute will be paid to M. Freyssinet, who is considered to be the real pioneer of prestressed concrete. The languages of the congress will be French, English, and Dutch. Papers are invited on any aspect of the design and construction of prestressed concrete. Manuscripts submitted before April 15, 1951, will be printed and distributed before the congress. Further information may be obtained from the secretary, M. F. G. Riessauw, 59 rue neuve St. Pierre, Ghent, Belgium.

A Continuous Prestressed Concrete Girder Bridge in France.

A ROAD bridge (*Fig.* 1) constructed during 1949 at Neufchâtel-en-Bray, France, is 154 ft. long and is a continuous prestressed concrete girder structure of three spans across a stream, a double-track railway, and two roads. The two outer spans (*Fig.* 2) are each 59 ft. long and the middle span 36 ft. The bridge carries a road 29 ft. 6 in. wide and two footpaths each 2 ft. 10½ in. wide. The constructional depth is about 2 ft.

on bogies (*Fig.* 7), enabling it to be moved laterally across the bridge.

When all the precast prestressed concrete beams were in position, transverse cables, (A) in *Fig.* 3, each cable containing twelve wires of 0·2 in. diameter, were threaded through holes in the beams, the anchorages for these cables being in the parapet beams. The gaps between the flanges of adjacent beams were filled with concrete, (E) in *Fig.* 3, and the transverse

Fig. 1.—Bridge at Neufchâtel.

Construction.

In each span there are twenty-two precast beams which, before erection, were prestressed to act as simply-supported beams carrying only themselves. The prestressing cables are of the Freyssinet type and each contains twelve wires of 0·2 in. diameter sheathed in bituminous paper. Rapid-hardening Portland cement was used and the cables, which were laid in the moulds before the concrete was placed, were stretched four days after casting the beams. The beams for the middle span (*Figs.* 4 and 5) were lifted into position by a locomotive crane. The beams for the end spans were cast on an approach ramp and, while slung one at a time from a steel beam, were slid into place across the gap between the two piers (*Fig.* 6). The steel beam was mounted

cables (A) were stretched and anchored so as to produce a transverse compression.

The continuity between the three spans was effected as follows. The beams across the middle span have no top flange (except the parapet beams) and the bottom flange is curtailed a short distance from each end. The beams over the end spans have a top flange throughout except for a distance of about 12 ft. from the end supported on the inner piers. For this distance only there is a top flange, as is seen in *Fig.* 7. Concrete blocks, (F) in *Fig.* 2, in which anchorage cones are embedded, were cast between the beams at the change of section and, when the blocks had hardened, three longitudinal cables, (D) in *Fig.* 2, of twelve 0·2-in. wires extending between the anchorages (F) were placed across the middle span in each space between the beams. Transverse

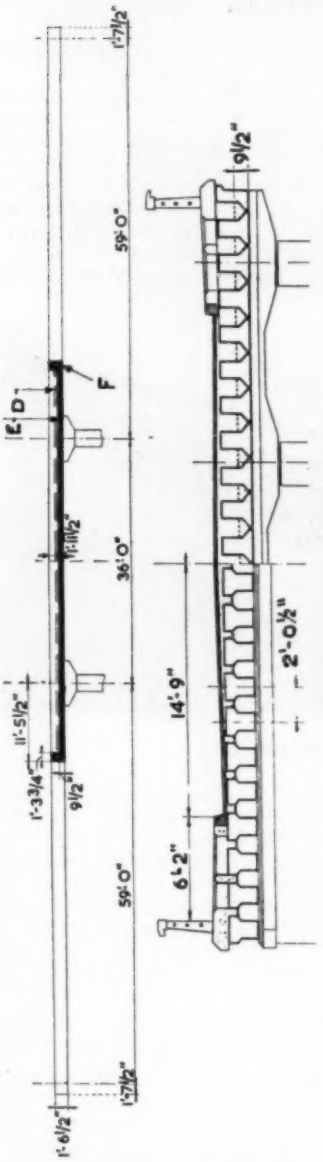

Fig. 2.—Longitudinal and Transverse Sections.

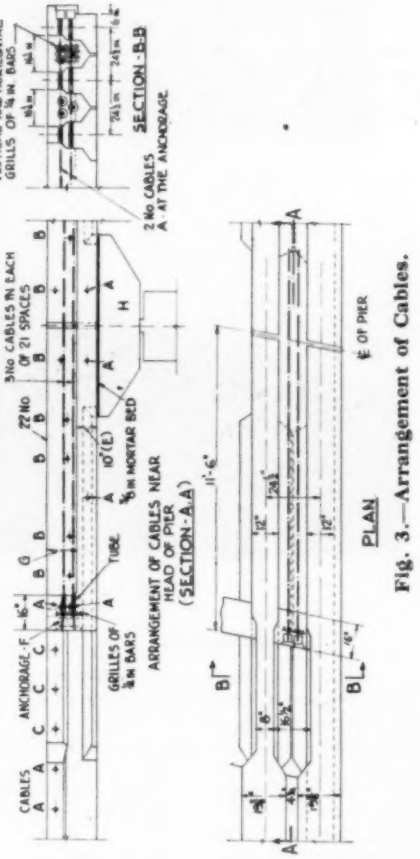

Fig. 3.—Arrangement of Cables.

cables (B), similar to (A), were placed next, and cables (D) were then stretched and anchored, thus combining the simply-supported beams into a structure continuous over three spans. The spaces (G) between the webs of the beams in the central part of the bridge were then filled with concrete, thus encasing the cables (D). The transverse cables (C), also similar to (A), were then placed and surrounded with concrete, and cables (B) and (C) were stretched and anchored, thus making the deck a fully prestressed slab. The reinforced concrete caps (H) of the middle piers add to the structural depth of the beams at the intermediate supports, thereby increasing the resistance to negative bending moment.

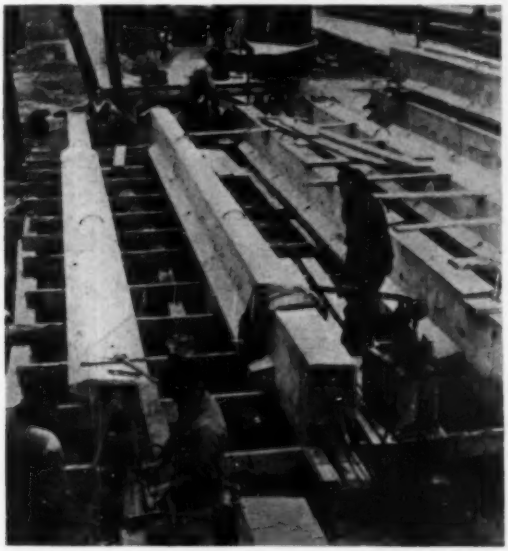

Fig. 4.—Prestressing Beams for Middle Span.

Design.

The concrete in the structure was vibrated and had a minimum strength of 6750 lb. per square inch at 90 days. The behaviour of the bridge under test load agreed very closely with that expected from the calculations. The total quantity of concrete is 413 cu. yd., and the structure contains 25·6 tons of high-tensile steel and 2·4 tons of mild steel. The equivalent elastic limit of the high-tensile steel is 178,000 lb. per square inch and the tensile strength 200,000 lb. per square inch. The working tensile stress in the wires after deductions for losses is assumed to be 121,000 lb. and 128,000 lb. per square inch. The permissible stresses in the concrete are given in *Table* I.

LOADS.—In addition to the weight of the structure at each stage of construction, the bridge is designed to carry an ordinary live load, which is briefly either a uniformly-distributed load of 230 lb. per square foot on the roadway and 87 lb. per square foot on the footpaths, or a train of six lorries each of 25 tons on the end spans

TABLE I.—PERMISSIBLE STRESSES (lb. per square inch).

	Maximum	Minimum
Beams during manufacture . . .	2420	142 (tension)
Before construction of superstructure	2140	0
Completed structure:		
No live load .	1890	142 (compression)
Ordinary live load	1890	142 ,,
Military live load	2420	142 (tension)

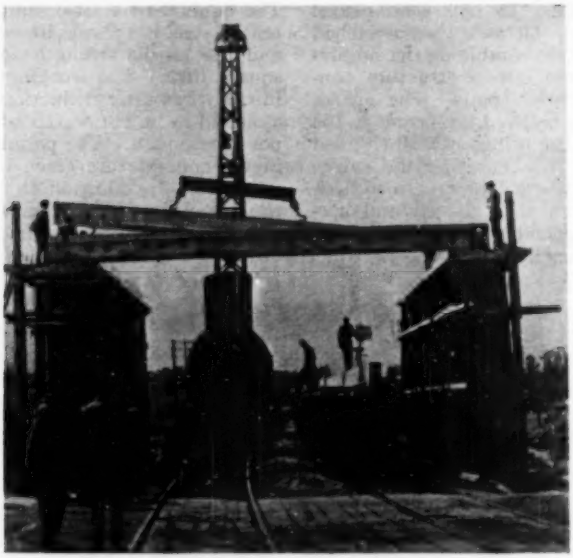

Fig. 5.—Erecting Beams for Middle Span.

and three similar lorries plus three axle-loads of 10 tons each on the central span. The alternative military loading is 80 tons on the roadway of the end spans and 75 tons on the middle span; this loading includes a vehicle of 70 tons. The live loads are multiplied by from 1·13 to 1·19 to allow for impact.

BENDING MOMENTS.—The average bending moments and shearing forces at six critical sections are calculated for the total width of the deck assuming freely-supported spans, and are increased to allow for some of the twenty-two beams being subjected to greater bending moments than others. The theories of M. Guyon ("Annales des Ponts et Chaussees," September and October, 1946) are applied to determine the increase and to calculate the transverse bending moments due to various positions of the live load. It is assumed that, in order to allow for

Fig. 6.—Method of Erecting End Spans.

subsequent continuity, the end spans are 90 per cent. and the central span 80 per cent. of their true lengths. The average bending moments are increased by 1·24 for the ordinary live load on beams in the central part of the deck, and 1·38 for the military load on beams at about the third points of the width of the deck when the 70-tons vehicle is placed eccentrically on the bridge.

STRESSES.—The areas and section moduli of three of the critical sections, namely, about the midspan of the end span, the middle of the central span, and the centre-line of the intermediate support, are given in *Table* II. The bending moments at these sections due to various conditions of loading during construction and the consequent stresses are also given. The effect of the prestress is given separately for the initial condition of the beams considered as freely supported and when the force in the continuity cables is acting. The stress due to the initial prestressing forces P is based on a direct compression P and a moment equal to P multiplied by the eccentricity of the cables at the section at different stages of construction. At the middle of the end spans the force in the cables is about 137 tons, the eccentricity is almost 9 in., and the bending moment is therefore 2,780,000 in.-lb. The corresponding values at the middle of the central span are about 98 tons, $2\frac{3}{4}$ in., and 522,000 in.-lb.

The total force P_c in the 63 cables provided for continuity over the intermediate supports is about 1310 tons, and the secondary (or supplementary) bending moment due to this force over the central span and parts of the end spans (*Fig.* 2) is about 5,650,000 in.-lb. The method of calculating this bending moment is given later.

The corresponding moments and consequent stresses due to prestressing alone are given in *Table* II. When the stresses due to the weight of the structure, the live loads, and the prestressing are known, the critical stresses are calculated by combining the component stresses in the most adverse manner at each stage of

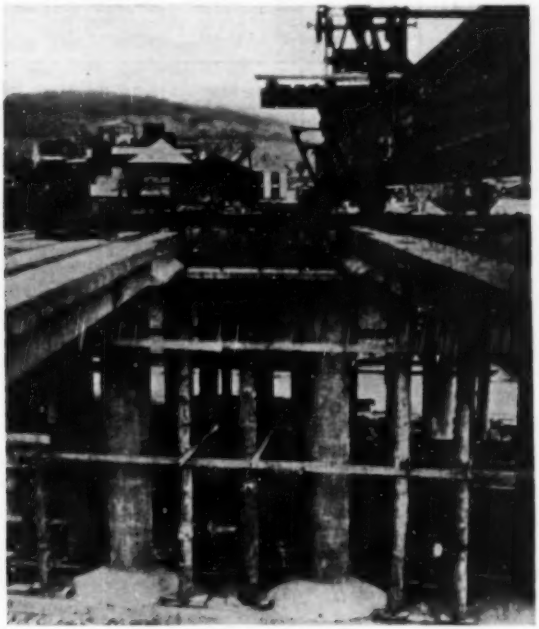

Fig. 7.—Erecting Beams for End Spans.

TABLE II.—COMPONENT AND RESULTANT STRESSES.

Sections	Middle of end span			Middle of central span			Intermediate support		
Properties of cross sections	One beam as cast	Full width of deck as completed		One beam as cast	Full width of deck: Bottom flange cast	Full width of deck: Complete			
Cross sectional area (sq. in.)	310	8320		333	8600	12,350	16,650		
Modulus of section (in.³):									
With reference to top	$1{\cdot}34\times10^3$	$36{\cdot}6\times10^3$		$1{\cdot}27\times10^3$	$28{\cdot}3\times10^3$	$48{\cdot}5\times10^3$	$85{\cdot}8\times10^3$		
With reference to bottom	$1{\cdot}03\times10^3$	$25{\cdot}9\times10^3$		$1{\cdot}63\times10^3$	$40{\cdot}0\times10^3$	$48{\cdot}5\times10^3$	$93{\cdot}4\times10^3$		
Component stresses (lb. per sq. in.)	Bending moment (in.-lb. × 10³)	Stress		Bending moment (in.-lb. × 10³)	Stress		Bending moment (in.-lb. × 10³)	Stress	
		Top	Bottom		Top	Bottom		Top	Bottom
Weight of beam (1)	+1480	+1103	−1422	+493	+385	−299	—	—	—
Cast-in-situ concrete supported by separate beams (2)	+230	+171	−222	+98	+77	−59	—	—	—
Ditto supported by 22 beams (3)	+65	+1	−3	+3440	+121	−85	−3300	−38	+36
Superstructure complete (4)	+10,400	+282	−398	−4030	−82	+82	−8900	−102	+92
Ordinary live load :									
Positive B.M. (5)	+33,700	+913	−1294	+23,600	+48	−48	+4340	+50	−43
Negative B.M. (6)	−3090	−84	+119	−15,100	−533	+377	−31,300	−361	+331
Military live load :									
Positive B.M. (7)	+41,400	+1121	−1636	+18,700	+384	−384	+4260	+50	−46
Negative B.M. (8)	−3460	−95	+134	−22,100	−777	+550	−38,200	−444	+408
Prestress :									
Initial (beams simply supported) (9)	−2780	−1074	+3670	−522	+256	+981	—	—	—
Final(continuity)(10)	−2670	−73	+102	+960	+377	+319	+26,700	+498	−85
Maximum resultant stresses (lb. per sq. in.)	Summation	Top	Bottom	Summation	Top	Bottom	Summation	Top	Bottom
Due to dead load and prestress :									
Compressive (A)	(1, 2, 3, 4, 9, 10)	+397	—	(1, 2, 3, 9, 10)	+1216	—	(3, 10)	+459	—
(B)	(1, 9)	—	+2247	(1, 2, 3, 10)	—	+942	(3, 4, 10)	—	+43
Tensile (C)	—	—	—	—	—	—	—	—	—
(D)	—	—	—	—	—	—	(10)	—	−85
Due to dead and live loads and prestress :									
Compressive (E)	(A, 7)	+1518	—	(A, 4, 5, 6, 7)	+1518	—	(3, 4, 5, 10)	+407	—
(F)	(B, 2, 3, 4, 8, 10)	—	+1861	(B, 3, 4, 5, 6, 7, 8)	—	+1434	(3, 4, 8, 10)	—	+451
Tensile (G)	—	—	—	—	—	—	(3, 4, 8, 10)	−87	—
(H)	—	—	—	—	—	—	(3, 4, 7, 10)	—	−3

construction and on completion. The maximum combined compressive and tensile stresses are also given in *Table* II. It is seen that the greatest compressive stress, 2247 lb. per square inch, is caused by the dead load and initial prestress. This stress occurs at the bottom of the beams at the middle of the end spans and is due solely to the initial prestress and the weight of the beam. The greatest stress produced by the military live load combined with the dead load and final prestress may be 1861 lb. per square inch, and occurs at the bottom of the deck at the middle of the end span. A tensile stress of 85 lb. per square inch due to the dead loads and the final prestress occurs at the bottom of the part of the head of the intermediate pier cast monolithically with the deck and, due to the dead and live loads and the final prestress, there is a tensile stress of 87 lb. per square inch

at the top of this part. These stresses should be compared with the permissible stresses given in *Table* I. The conversions from the metric system of the data in *Table* II are approximate.

The combination of the compressive and shearing stresses due to the dead and live loads and the final prestress cause practically no principal tensile stresses.

SECONDARY BENDING MOMENTS.—The method of determining the secondary bending moments M_s at the intermediate supports is given in the following calculations; these are based on metric units and the conversions are approximate.

Assume that l_1 and l_2 are the lengths of the end spans and central span respectively, e the eccentricity generally, e_1 the eccentricity of the cables at the midspans of the end and central span, e_2 the eccentricity for a length of about 0·6 m. (2 ft.) on each side of the intermediate supports, I the moment of inertia generally, and I_1, I_2, and I_3 the moments of inertia respectively at the midspan of the end spans, at the midspan of the central span, and for a length of 0·6 m. (2 ft.) on each side of the intermediate supports. The moments of the prestressing force P_c in the continuity cables are $P_c e_1$ and $P_c e_2$. The equation of bending moments is

$$\left(\frac{\Sigma_1}{l_1^2}+\frac{\Sigma_2}{l_2^2}+\frac{\Sigma_3}{l_2^2}\right)M_s = -\left(\frac{\Sigma_4}{l_1}+\frac{\Sigma_5}{l_2}\right)P_c,$$

the terms in which are values of $\frac{A\bar{x}}{Il}$, and the summations are therefore

$$\Sigma_1 = \int_0^{l_1}\frac{x^2dx}{I};\quad \Sigma_2 = \int_0^{l_2}\frac{(l-x)^2dx}{I};$$

$$\Sigma_3 = \int_0^{l_2}\frac{x(l-x)dx}{I};\quad \Sigma_4 = \int_0^{l_1}\frac{ex.dx}{I};$$

$$\text{and } \Sigma_5 = \int_0^{l_2}\frac{e(l_2-x)dx}{I}.$$

In each span, x is measured towards the right from the support. The moments of inertia are: I_1, 0·130 m.^{4} (312,000 in.^{4}); I_2, 0·163 m.^{4} (391,000 in.^{4}); I_3, 0·807 m.^{4} (1,940,000 in.^{4}). The effective lengths of the spans are: l_1, 18 m. (59 ft.); l_2, 11 m. (36 ft.). Substituting these values and integrating, the terms in the left-hand side of the bending-moment equation have the following values (in metric units): $\frac{\Sigma_1}{l_1^2} = 42{\cdot}3$; $\frac{\Sigma_2}{l_2^2} = 34$; and $\frac{\Sigma_3}{l_2^2} = 11{\cdot}1$. Therefore the left-hand side of the bending-moment equation equals $87{\cdot}4M_s$.

Considering the right-hand side of the equation, it must be noted that P_c commences to act at the section where $x = 14{\cdot}5$ m. (47·6 ft.) in the end span. By integrating (in metric units)

$$\frac{\Sigma_4}{l_1} = 16{\cdot}5e_1 + 0{\cdot}730e_2.$$

$$\frac{\Sigma_5}{l_2} = 43{\cdot}3e_1 + 1{\cdot}445e_2.$$

$$\frac{\Sigma_4}{l_1}+\frac{\Sigma_5}{l_2} = 59{\cdot}8e_1 + 2{\cdot}175e_2.$$

Since e_2 is 0·282 m. (11¼ in.) and e_1 is $e_2 - 0{\cdot}222$ m. (that is $e_1 - e_2 = 8\frac{3}{4}$ in.), substitution in the bending-moment equation gives

$$87{\cdot}4M_s = -[59{\cdot}8(e_2 - 0{\cdot}222) + 2{\cdot}175e_2]P_c,$$

and, since P_c is 1336 metric tons (1310 tons), the secondary bending moment is given by

$$M_s = -1336[(0{\cdot}71 \times 0{\cdot}282) - 0{\cdot}152]$$
$$= -65 \text{ tons-metres},$$

that is 5,650,000 in.-lb.

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Piles 87 ft. Long at Plymouth.

For the foundations of part of the new coaling wharf at Plymouth "B" power station, now in course of construction for the British Electricity Authority, there are precast reinforced concrete piles 87 ft. long. The wharf, which replaces an old timber wharf on the same site, is a reinforced concrete structure 575 ft. long supported on concrete-filled cylindrical piers 3 ft. 6 in. in diameter and precast concrete piles 18 in. square. The piers bear on a hard limestone with an irregular surface, which forms the head of a subterranean cliff the face of which crosses the site of the wharf. It is therefore necessary for part of the wharf to be supported on foundations bearing on the foot of the cliff, and it is here that the long piles were required.

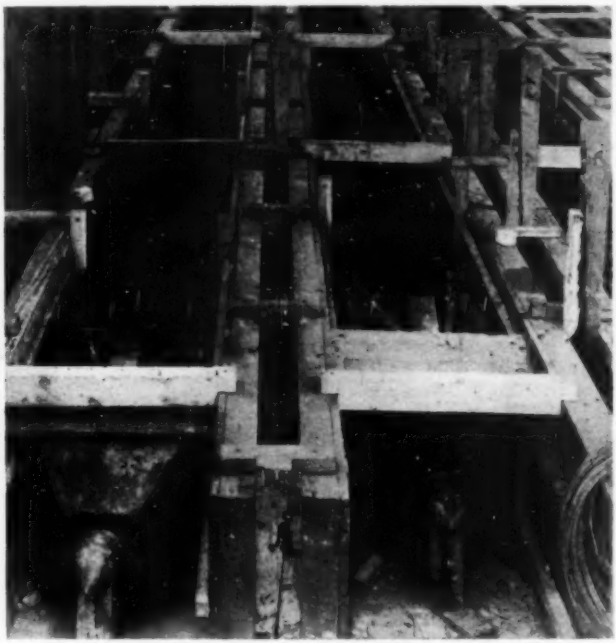

Fig. 1.—Moulds for Piles.

The piles (*Fig. 2*), of which there are 33, are 21 in. square and, although they were 87 ft. long when driven, they were lengthened in situ to 107 ft. in some cases. Cast-iron rock-type shoes, each about 5 cwt. in weight, were provided. The main reinforcement is four 1¼-in. bars, each in two or more lengths connected by a double-V butt weld. The welds are staggered so that no two occur at one cross section and each is strengthened by two 1-in. splice bars (*A*) 4 ft. 6 in. long. The links are ¼ in. diameter and are generally at 6-in. centres except near the ends of the pile where the spacing is 3 in. For a length of 71 ft. 6 in. commencing 7 ft. from the head the pile is hollow, the cavity being formed by 14-in. diameter No. 18-gauge steel tubes (*B*) in lengths of 4 ft. The tubes are connected by an internal coupler (*C*). In the solid parts of the pile ordinary steel forks (*E*) are provided to retain the bars in position, but where the cavity occurs two sets of four projecting metal lugs (*D*) are welded

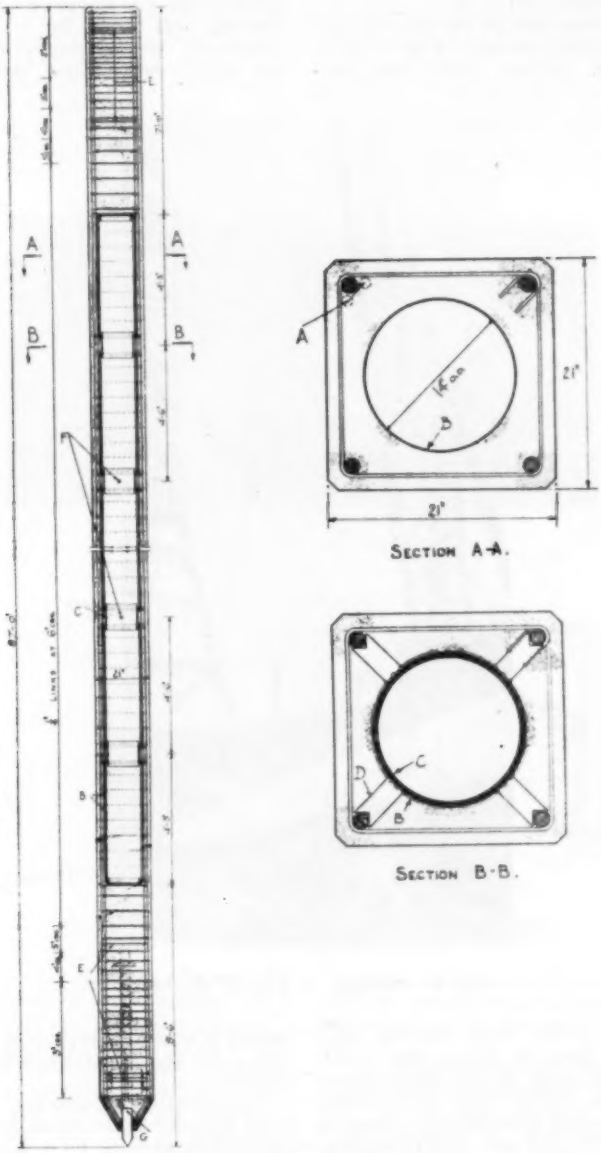

Fig. 2.

on the couplers. The cover of concrete outside the bars is 1½ in. The mixture of the concrete is 1 : 1 : 2.

The piles were cast on the site five at a time in wooden moulds (*Fig.* 1) in a covered bay. Lifting holes were provided 17 ft. 3 in. from the tip and 15 ft. 9 in. from the head, that is at about 0·2 and 0·185 of the length from the respective ends. The holes were formed by welding wrought-iron tubes of 1½-in. bore (*F*) across two of the couplers, which were drilled accordingly. The piles were lifted (*Fig.* 3) by a 15-tons derrick crane after having been brought within reach of the pile frame by a mobile crane. Driving was done by a 5-tons semi-automatic steam-hammer operating on an 85-ft. steel frame with extended leaders (*Fig.* 3). The drop of the hammer was 4 ft. and the piles were driven to refusal.

Fig. 3.—Slinging a Pile 87 ft. Long.

The consulting engineers are Messrs. L. G. Mouchel & Partners, Ltd., and the contractors Messrs. John Laing & Son, Ltd.

Reinforced Concrete Grandstands.

Two unusual reinforced concrete grandstands are shown in the accompanying illustrations. *Figs.* 1 and 2 show a grandstand incorporating a cantilevered shell roof recently erected at Madrid, and *Figs.* 3 and 4 give details of a similar structure at Cartagena, Columbia.

The structure at Madrid (*Fig.* 1) is described in an abstract from "La Revista Nacional de Arquitectura" in a recent number of "Die Bautechnik". The seats or steps are supported on arches of 16 ft. 6 in. span, which are continued to the rear to support the ceiling of the great hall. The roof over the seating is a hyperboloidal shell and cantilevers a distance of 42 ft. beyond the columns, which are immediately behind the tiers of seats (*Fig.* 1). The structural element in this roof is a "butterfly" member comprising a central rib from which spring two semi-arches. Adjacent semi-arches meet along a sloping line. The rise of each semi-arch is 4 ft. 7 in. above the gutter at the column head, at which point the thickness of the slab forming the semi-arch is 6 in. At the front edge the rise is 1 ft. 7½ in. and the slab is 2 in. thick. The back part of the roof is supported on two rows of columns and anchored at the edge by vertical tie-rods 16 ft. 6 in. apart and connected to the main frames of the hall.

Fig. 1.—Grandstand at Madrid.

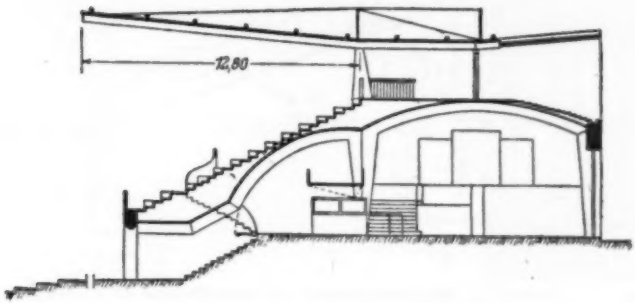

Fig. 2.—Cross Section of Grandstand at Madrid.

The details of the reinforcement in the grandstand at Cartagena are shown in *Fig.* 4 (reproduced from "Beton-und Stahlbetonbau"). The sizes of the bars are in millimetres (25 mm. = 1 in.). The construction occupied 160 days. The seats are Z-shaped beams supported on the lower part of the main beams, which are curved parabolically like a letter C

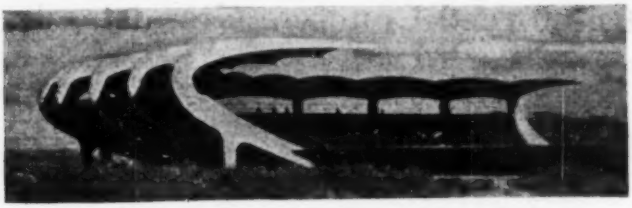

Fig. 3.—Grandstand at Cartagena.

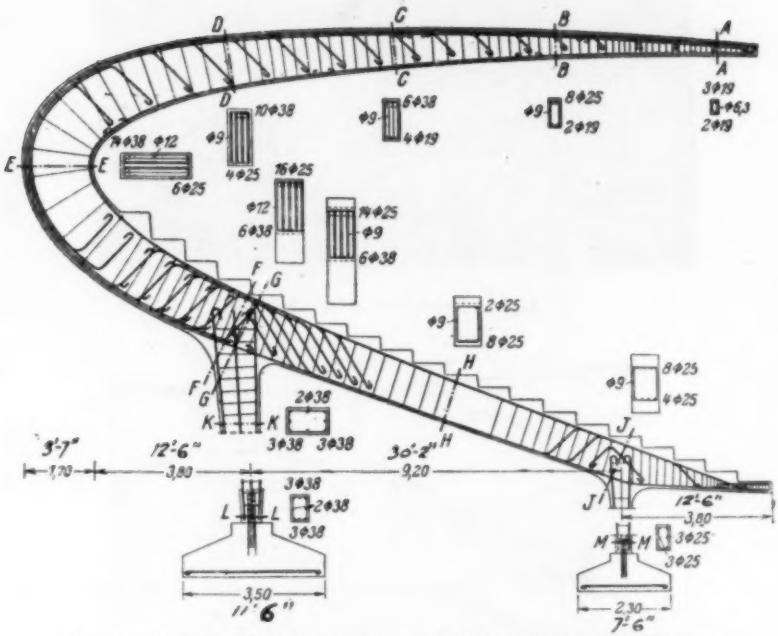

Fig. 4.—Details of Reinforcement of Grandstand at Cartagena.

and are at 35 ft. centres. The roof slab is a reinforced concrete shell having a least thickness of 1½ in. (4 cm.). Reinforced concrete was used for this structure mainly because of the large salt-content of the sea-air. The concrete in the frames had a compressive strength of 2000 lb. and that in the shell 3000 lb. per square inch.

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Reconstruction of Quay Wall, Rotterdam.

THE accompanying diagrams, which are from "Cement" (Amsterdam), show the principal stages in the reconstruction of a damaged reinforced concrete quay wall in the harbour at Rotterdam. The wall, shown at (X) in *Fig.* 1, is 410 ft. long and is of counterfort construction supported on a slab carried on inclined and vertical piles. The stages of the construction (*Fig.* 1) are: (I) A row of steel sheet-piles is driven behind what remains of the original slab and a layer of sand is placed on the bed of the harbour; (II) Temporary piles to support the timbering are driven from a light floating piling-frame, and a king-pile to support the end of a temporary gantry is driven from a heavier floating piling-frame; (III) A row of steel sheet-piles is driven by the heavier floating crane in front of

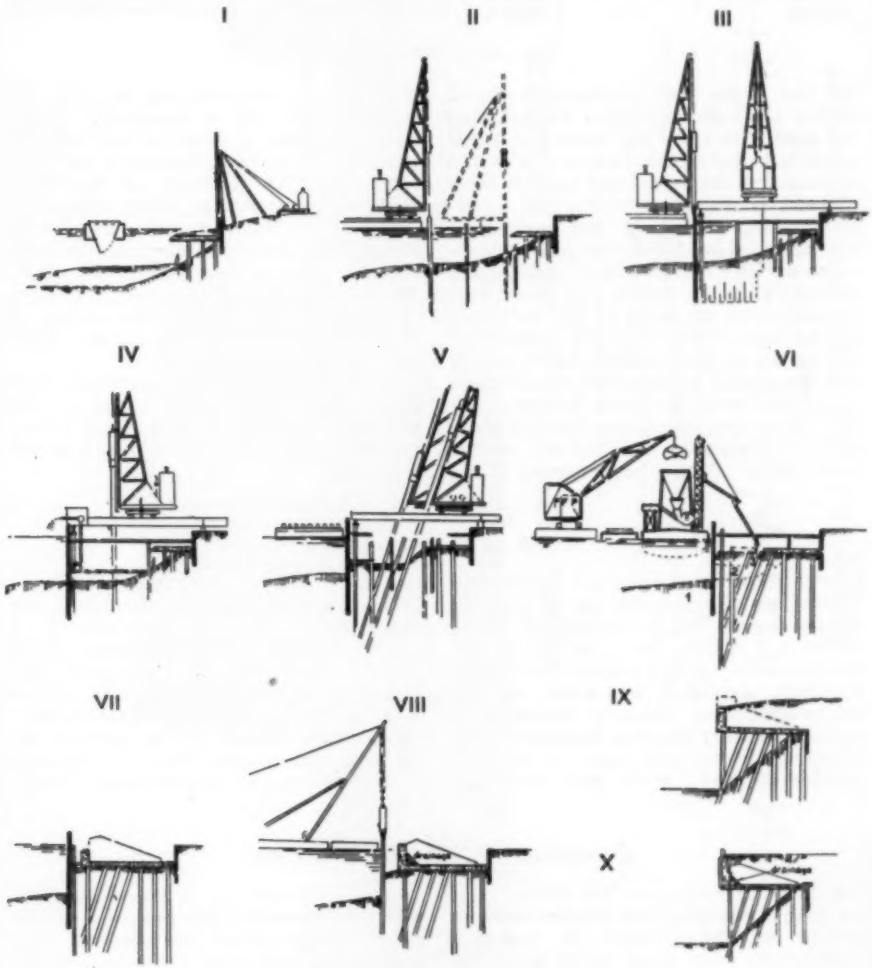


Fig. 1.—Stages in the Work.

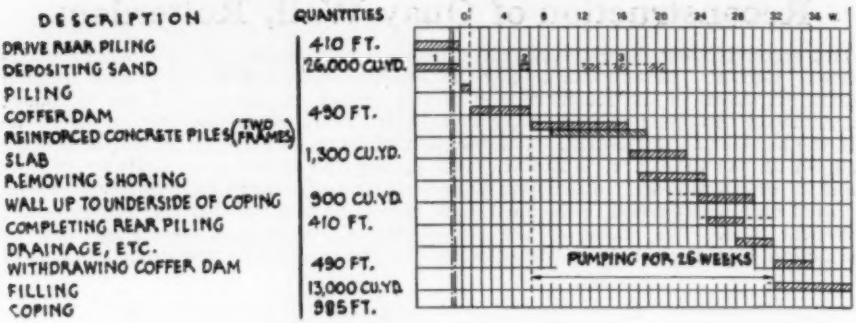


Fig. 2.—Time Schedule.

the site of the wall, timbering is placed within the cofferdam thus formed, and the gantry to carry the main pile-driver is erected over the cofferdam. (IV) The cofferdam is dewatered and another layer of sand is deposited within the cofferdam; the driving of the permanent vertical reinforced concrete piles is commenced. (V) The remaining vertical and inclined piles are driven and more sand is laid up to the level of the underside of the slab. (VI) Concrete is batched and mixed in plant erected on a pontoon and is supplied by a floating crane which grabs the materials from barges; the concrete is distributed by a chuting plant and is deposited to form the slab on the sand filling. (VII) The timbering in the cofferdam is removed and the first lift of the front wall is concreted. (VIII) The sheet-piles in front of the wall are extracted and the counterforts are constructed. (IX) Removal of the front sheet-piles permits the filling within the cofferdam to fall and attain its natural slope; the sheet-piles behind the wall are cut down to the level of the slab; filling is deposited above the slab to form the deck of the quay. The coping of the wall is constructed and the work completed.

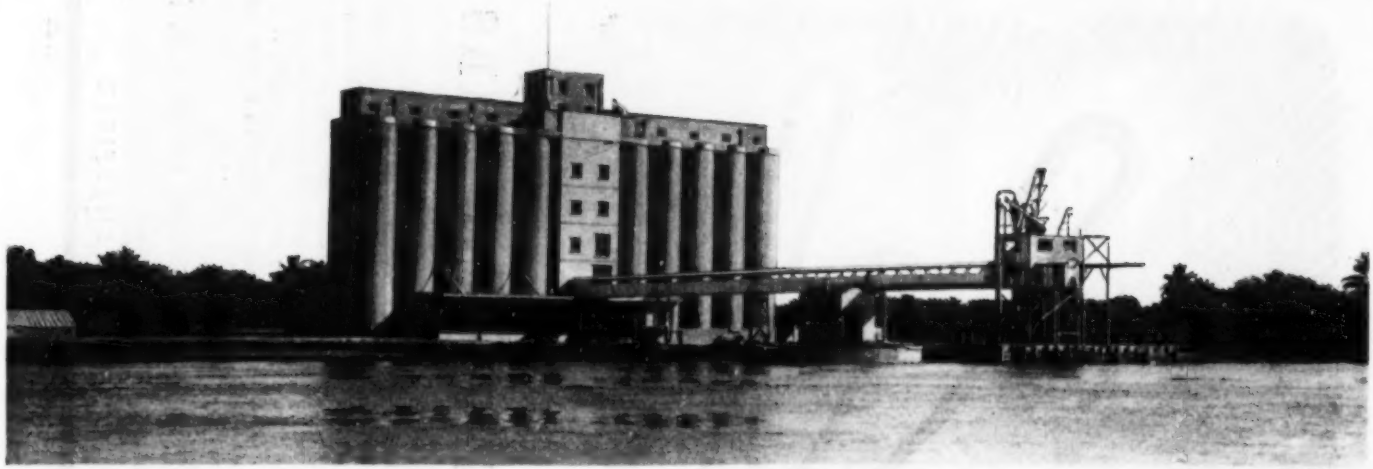
The time-schedule for the construction of the quay wall is shown in *Fig.* 2. All preparatory work was completed before the beginning of the period shown on the diagram.

Wood as Reinforcement.

In reference to a note in our number for September, 1950, on the use of wood as reinforcement, Mr. A. Badian, of Haifa, writes: "About the year 1913 Dr. Marcichowski, of the Szkola University, Lemberg, published a booklet on the reinforcement of concrete beams with wood. In 1919 Dr. von Emperger published in 'Beton und Eisen' a series of articles entitled 'Holz und Schilf als Ersatz des Eisens in der Zugbewehrung' ('Wood and Reeds as a Substitute for Steel for Tensile Reinforcement'). I have no doubt that the suggestion, which has now been revived in Hungary, probably due to the shortage of steel, will be found once more to be impracticable." [It is reported that one at least of the pioneers of reinforced concrete in France first used wood as reinforcement.—Ed.]

Consultant in Precast Concrete.

Mr. R. H. Baumgarten has retired from his post in charge of the precast concrete section of the Cement & Concrete Association. For many years before he joined the Association, Mr. Baumgarten was well known as a consultant and technical adviser in the precast concrete industry, and he has now resumed independent practice at 90 Thornton Avenue, London, W.4.



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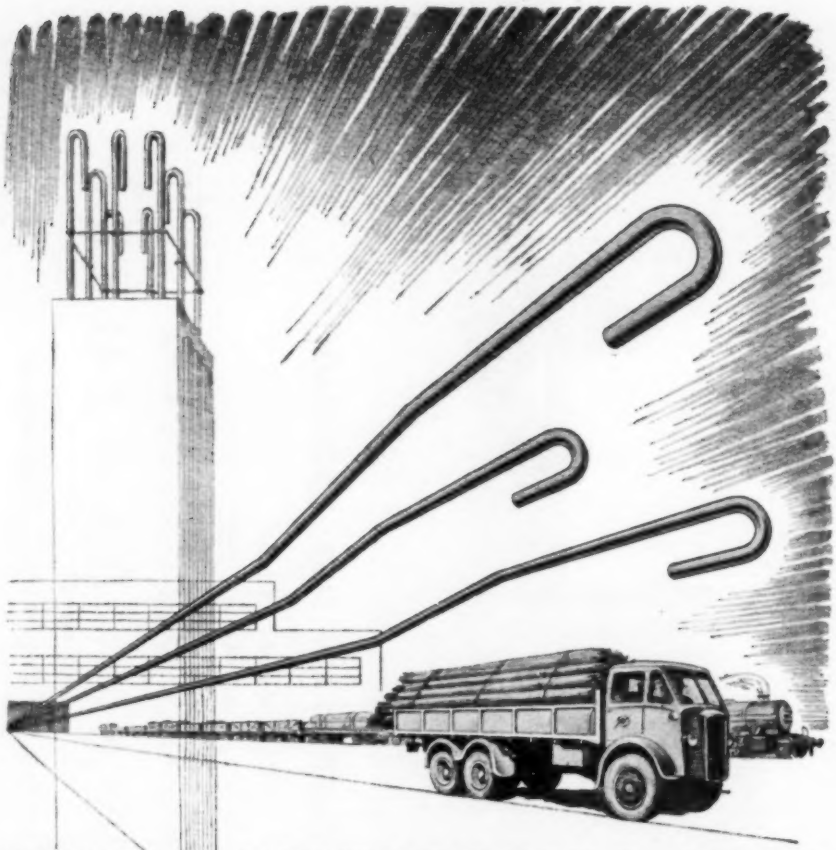

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An Unusual Reinforced Concrete Building.

THE fifteen-story reinforced concrete tower illustrated in *Figs.* 1 to 3 (from "Engineering News-Record") is a research laboratory and is 154 ft. high. The floors cantilever from a central well of 13 ft. minimum diameter. The nine main floors (*Fig.* 2) are 40 ft. square and the six intermediate floors are 38 ft. in diameter. The well contains lifts, stairs, and services. Except for a parapet of brick and concrete at each floor level, the walls are of glass between aluminium stanchions. The floors (*Fig.* 3) are of cellular construction, the top slab being

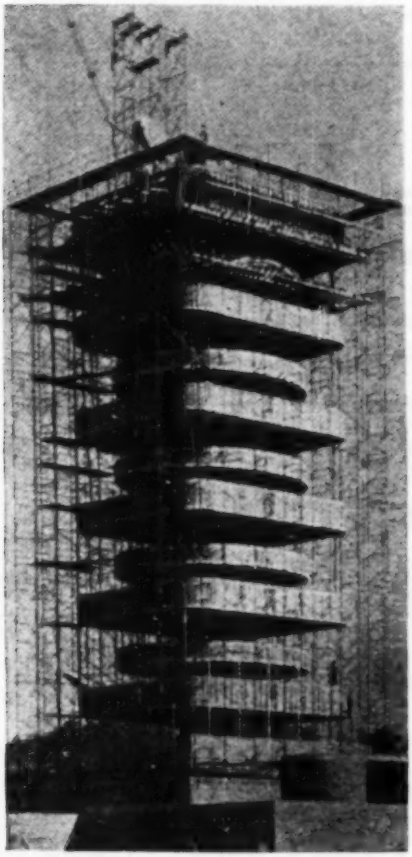

Fig. 1.—Structure During Construction.

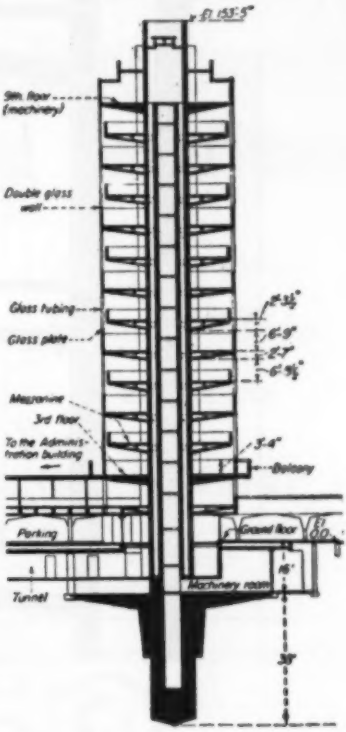

Fig. 2.—Cross Section.

2½ in. thick, and the bottom slabs varying from 10 in. at the well to 7½ in. at the parapet for the main floors and from 5 in. to 3½ in. for the intermediate floors. The main circumferential reinforcement bars in the slabs are lapped and welded. The secondary reinforcement in the slabs is expanded mesh welded at points of contact.

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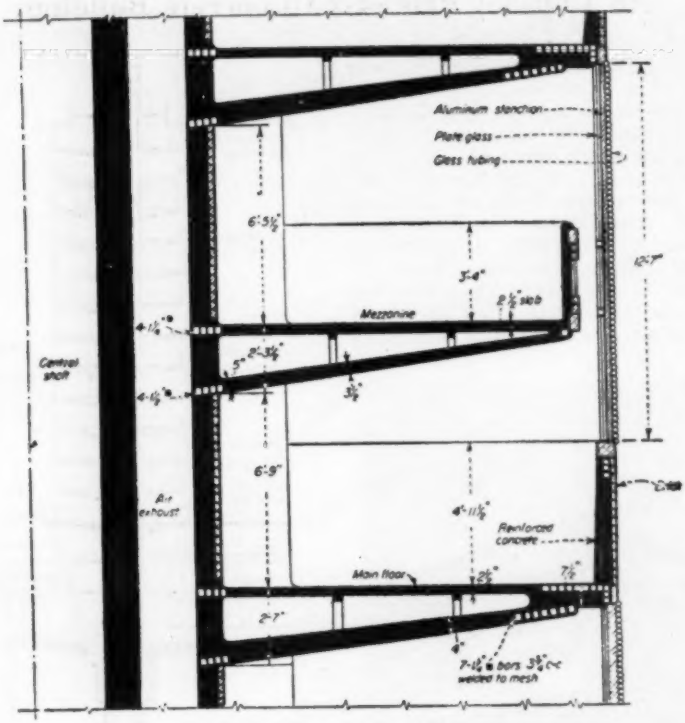

Fig. 3.—Details of Floors.

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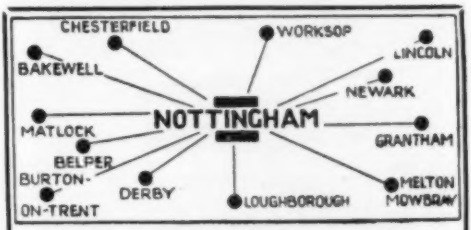

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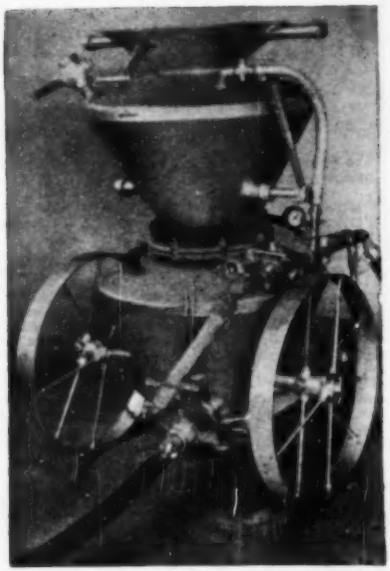

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AN 8-ft. horseshoe-shape sewer 3¼ miles long in Oregon is being lined with concrete which is brought into the tunnel through the manhole shafts which are at intervals of about 1000 ft. At the bottom of the shaft the concrete is placed in an agitator on a trolley, which is drawn by an electric locomotive to a pumping plant in the tunnel. The concrete is discharged on to an elevator that takes it to a hopper for remixing before it is fed into a concrete pump. The concrete is pumped through a pipeline 600 ft. long and deposited behind steel shutters. The thickness of the lining is about 9 in., and the lining of the walls and roof is completed before the invert is placed.

It is reported in "Engineering News-Record" that before this method was adopted some of the tunnel was lined by using two pumps, one immediately below a manhole shaft and the other starting near where the concrete was being placed and brought towards the shaft as the operation of placing moved in that direction. The pumps were connected by an 8-in. pipeline which at times was 1300 ft. long. This method was abandoned because of the difficulty of preventing plugs forming in the long pipeline.

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Colorcrete (buff, red, and khaki), in 6-ton loads, 110*s.* 6*d.*; paper bags 12*s.* extra.

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SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1*s.* 1*d.*) and No. 94 (price 5*d.*) issued by H.M. Stationery Office.

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(*Contracts up to £3000. Inc. 10 per cent. profit.*)

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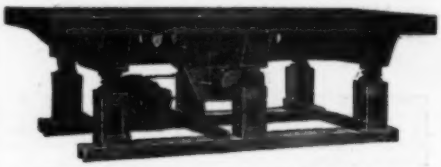

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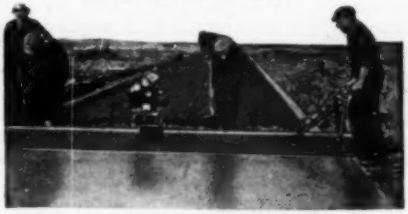

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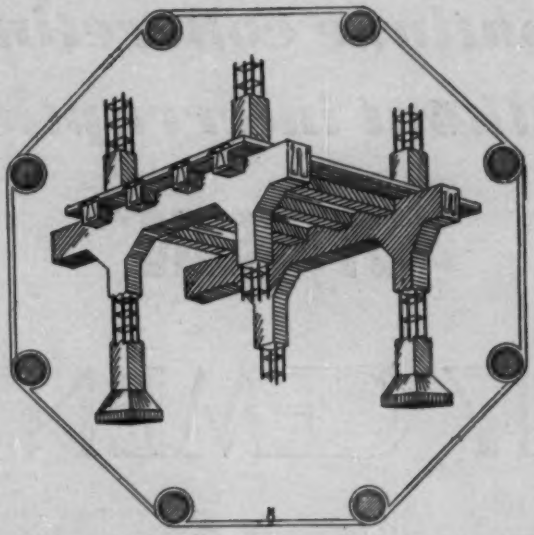

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